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GUIDELINES FOR RETROFITTING OF STEEL BRIDGES BY PRESTRESSING

INDIAN ROADS CONGRESS

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GUIDELINES FOR RETROFITTING OF STEEL BRIDGES BY PRESTRESSING

1. INTRODUCTION

1.1 The Guidelines for Prestressed Steel Bridges has been under the consideration of Steel Bridges Committee since March, 2003. A Sub-committee under the Convenorship of Shri B.C. Roy with Dr. H. Subba Rao, Dr. B.P. Bagish and Shri S. Chatterjee was constituted to draft the Guidelines. The draft Guidelines prepared by the Sub-committee was extensively deliberated in the various meeting of erstwhile Steel Bridges Committee (B-7) till December, 2005.

1.2 The Steel Bridges Committee was reconstituted in January, 2006 and retitled as Steel and Composite Structures Committee (B-5) of the Indian Roads Congress with following personnel:

Ghoshal, A.	*Convenor*
T.K. Bandyopadhyay, Dr.	*Co-Convenor*
Ghosh, U.K.	*Member-Secretary*

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1.3 The newly constituted Steel and Composite Structures Committee (B-5) in its meeting held on 12.5.2007 at Kolkata has finalised the draft "Guidelines for Design of Prestressed Steel Road Bridges" and recommended its placement before the Council through BS&S Committee.

1.4 The draft document "Guidelines for Retrofitting of Existing Prestressed Steel Road Bridges" was approved by the Bridges Specifications and Standards Committee in its meeting held on 3rd June 2007 and Executive Committee in its meeting held on 7th June, 2007. The document was approved by the IRC Council in its 182nd meeting held on 18th August, 2007 NITHE, Noida for printing subject to minor modifications and also changed the title of the document as "Guidelines for Retrofitting of Steel Bridges by Prestressing".

1.5 Most of Steel Bridges in India have been and are being constructed by the local authorities, State PWDs, National Highway Divisions, Indian Railways by using steel bridge code of Indian Railways and recommendations of IRC:24 Section V-Steel Road Bridges.

1.6 With the increasing requirement of large span superstructures over rivers, gorges or other locations where no intermediate support is available, prestressed steel bridge system may offer fast and economic solutions for vehicular as well as pedestrian bridges.

1.7 In view of the above it is felt necessary to develop a design guideline for prestressed steel bridges. Guidelines are intended to lay down the requirements for design and construction of prestressed steel road bridges.

1.8 Taking into consideration the fact that Prestressed Steel Road Bridges have not been used widely internationally and there are only few case studies available in the country, it is intended that this guideline be used only for retrofitting of existing steel road bridges, which are found to be in distress or which require further augmentation of capacity due to changed design consideration. Wide use of this guidelines for new bridge work will only be permitted after successful use of these guidelines for some more time.

2. DEFINITION OF PRESTRESSING

The term 'prestressing' in the text denotes application of predetermined concentric or eccentric force to a steel structure/member so that the state of stresses in the members resulting from the applied force counteracts the stresses due to other external loads and keeps the resultant stresses in all the members within specified limits.

3. SCOPE

Prestressing of steel bridges can be done by various means such as by pre-deflecting the structure, imposing intentional deflection at the supports, by lack of fit, and other methods mentioned in references (2) & (3).

However, these guidelines deal mainly with specific requirements of simply supported superstructures of prestressed steel bridges using prestressing tendons. These shall be considered as complementary to the requirements contained in IRC:24-2001. The types of bridges covered in this documents are:

3.1 For Deck type Bridges

3.1.1 Rolled beams for span upto 10.0m (say)

3.1.2 Propped rolled beams composite with RCC deck slab, post tensioned after maturity of RCC for span upto 15.0m (say).

3.1.3 Propped plate girders composite with RCC deck slab, post tensioned after maturity of RCC for span upto 40.0m (say)

3.2 For through type Bridges and Arch Bridges

3.2.1 Main trussed prestressed either with straight/bent cables, with/without mechanical deviators made of circular/sectoral pulleys with shafts, diaphragm struts, for 25.0 m to 120.0 m span (say).

3.2.2 Steel arch bridges, similarly prestressed as above, for 75 m to 150 m span (say).

Notes: (i) For bridges with carriageway consisting of more than 2 lanes, cross girders may also be prestressed.

(ii) For large spans, prestressing may be carried out in stages.

4. RELEVANT CODES

4.1 IRC Codes

(i) IRC:5-1998 (Section I General Features of Design)

(ii) IRC:6-2000 (Section II Loads & Stresses)

(iii) IRC:18-2000 (Prestressed Concrete-Road Bridges)

(iv) IRC:21-2000 (Section III Plain & Reinforced Concrete)

(v) IRC:22-1986 (Section VI- Composite Construction)

(vi) IRC:24-2001 (Section V Steel Road Bridges)

4.2 IS Codes

(i) IS:1343-1980 (Prestressed Concrete)

(ii) IS:800-1984 (Steel Code)

5. SYMBOLS

Symbols are as defined in the text. Otherwise the symbol as per IRC:24-2001 shall be followed for appropriate meaning indicated against them.

A	Area of cross section of member
A_1	Area of top flange
A_2	Area of bottom flange
A_t	Area of tendon
A_w	Area of web
E	Elastic modulus
E_m	Elastic modulus of member
E_t	Elastic modulus of tendon
F	Allowable stress in structural steel
F_t	Permissible stress of tendon Material
F_{total}	Design Force acting on the member
ΔF_{total}	Part of total force under external loading, acting in tendon
H	Tendon distance below center line of bottom chord of main truss
I	Moment of inertia
I_X	Moment of inertia of girder about neutral axis
K	h/t_w
L	Length of beam
L_t	Length of prestressing tendon
M	Bending moment due to external loading
P	Concentrated load
S	Section modulus of symmetrical section
S_1	Section modulus for compressed edge
S_2	Section modulus for tensioned edge
T	Period of natural vibration of girder
X	Prestressing force
ΔX	Self stressing force
Y_L	Total deflection due to dead load, imposed load and impact
Y_P	Total upward deflection due to prestressing
Z	A prestressing force
h and h_2	Top & bottom fibre distances from neutral axis
a	h_2/h parameter characterizing asymmetry of an I-girder
e	Eccentricity of tendon with respect to neutral axis girder
f_{bf}	resulting stress in tendon
f_c	compressive stress
f_m	allowable stress in structural steel
f_t	allowable stress in tendon
g	acceleration due to gravity
h	depth of web $\approx h_1 + h_2$
k	Ratio of force in tendon to force in truss member
m	A_w/A_m parameter characterizing material distribution
n	natural frequency of vibration of girder or, $= h/d$, d = thickness of symmetrical section
n_1	is an overloading factor
n_2	is an underloading factor

r	a ratio of areas in prestressed trusses
t_w	thickness of web
$x_1, x_2, x_3...$	points of application of concentrated loads
w	uniformly distributed load
ß	a ratio of increase in tendon prestress
$\delta_{Prestress}$	Upward deflection due to prestress
Ψ	Coefficient of buckling

Above symbols or any other symbol used elsewhere are defined in the corresponding section.

6. MATERIALS

6.1 **Prestressing steel conforming to Indian standard code shall be used in association with structural steel covered in IRC:24-2001**

(i) Plain hard drawn steel wire for prestressing conforming to IS:1785 (Part 1 & Part 2) - 1983

(ii) Cold drawn indented wire conforming to IS:6003-1983

(iii) High tensile bar conforming to IS:2090-1983

(iv) Uncoated stress relieved strand conforming to IS:6006-1983

(v) Low relaxation steel wires, tendons and cables for prestressing conforming to IS:14268-1995

6.2 **Ducts for grouting of prestressing tendons/cables**

(i) Steel tubes for structural purposes conforming to IS:1161-1998

(ii) Mild steel tubes conforming IS:1239 (part 1) - 1990

(iii) High density polyethylene (HDPE) pipes conforming to IS:8008 (Part 1)-1976, and meeting additional requirements of IRC:18-2000

Note: The steel tubes should be medium/heavy duty galvanized pipes

6.3 **Structural Steel**

As indicated in IRC:24-2001

6.4 **Fasteners**

As indicated in IRC:24-2001

6.5 **Welding**

As indicated in IRC:24-2001

6.6 **Castings and forgings**

As indicated in IRC:24:2001

7. GENERAL FORMS AND ARRANGEMENTS

Some typical cross sections with tendons and deviator details are shown below:

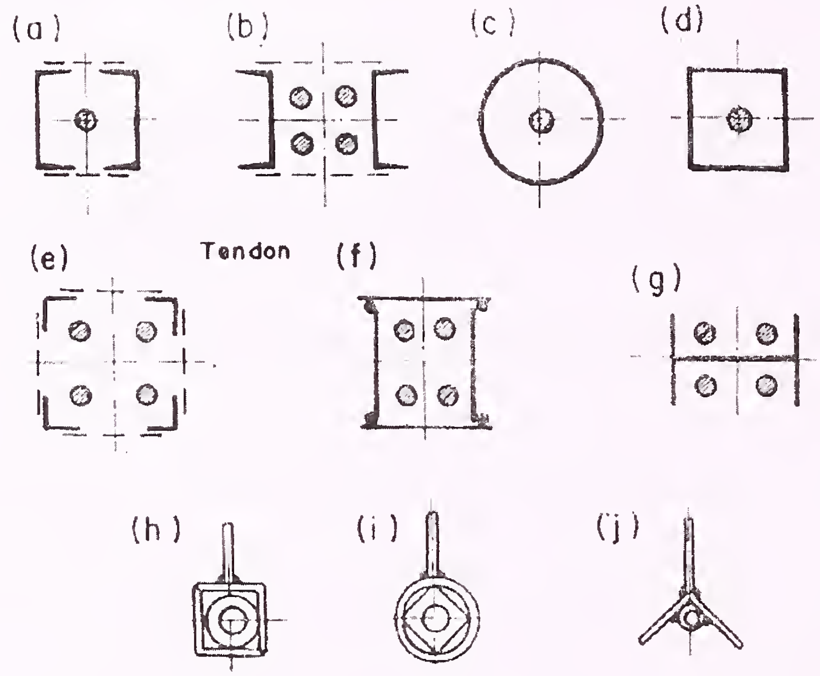


Fig. – 1 Types of cross sections of members with tendons

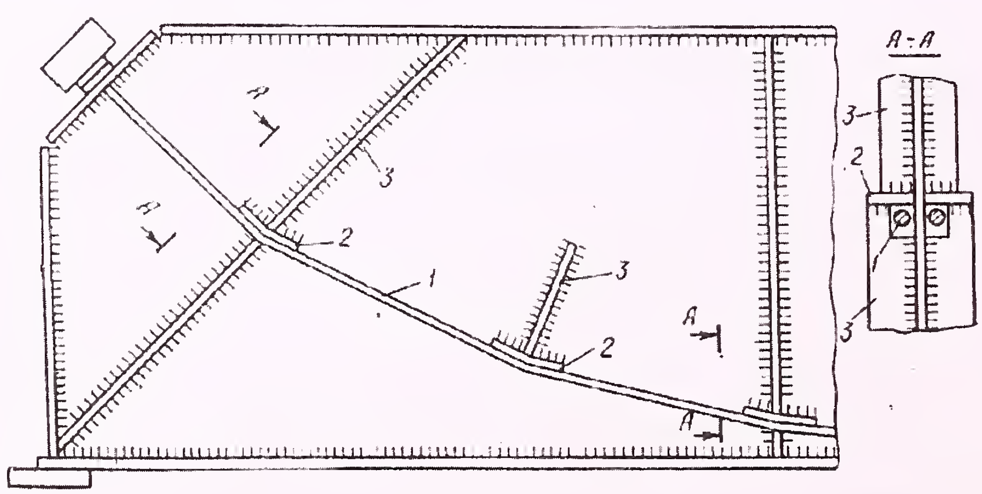


Fig. – 2 Guides for tendons of curvilinear outline
1 – Tendon : 2 – Guide : 3 – Rib

Following general forms and arrangement shall be used for beams/trusses/arches.

7.1 Beams

General profile of tendon is considered as straight in most of the cases. Bent cables can be used as when required. Fig.3(a) to Fig.3(f) and Fig.4(a) to Fig.4(c) show different patterns of girders. Tendons are shown in dotted lines.

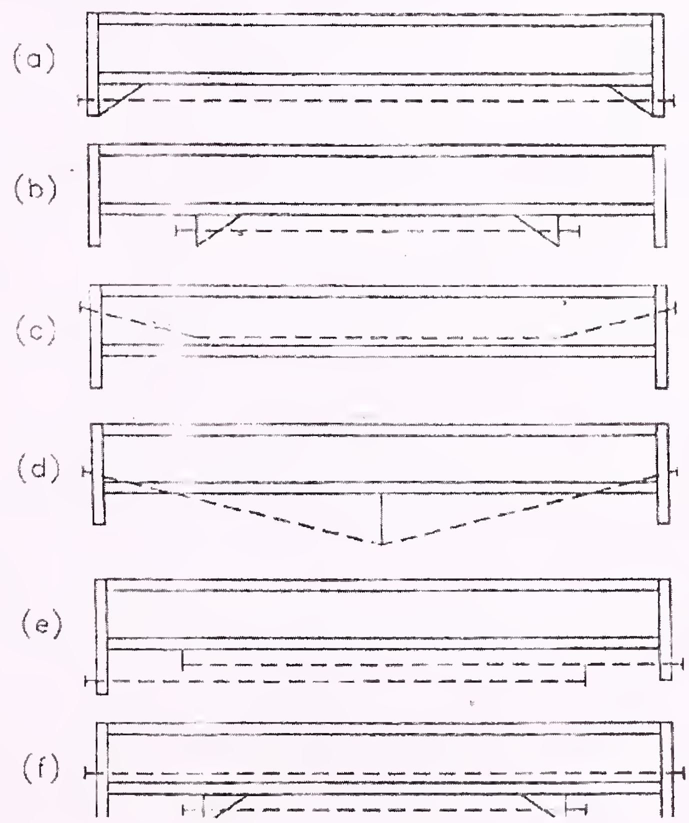


Fig. – 3 Location of tendons in single span beams

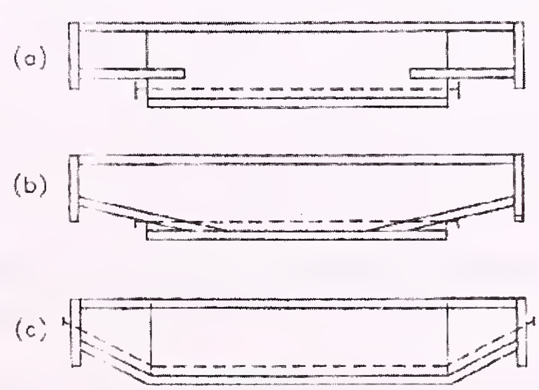


Fig. – 4 Location of tendons in beams of variable height

7.2 Trusses

Profile of tendon in main truss shall be either with straight cable inside the bottom chords or with additional bent cables below the bottom chords with mechanical diverters. Fig.3 to Fig.5 show different forms of prestressed steel trusses.

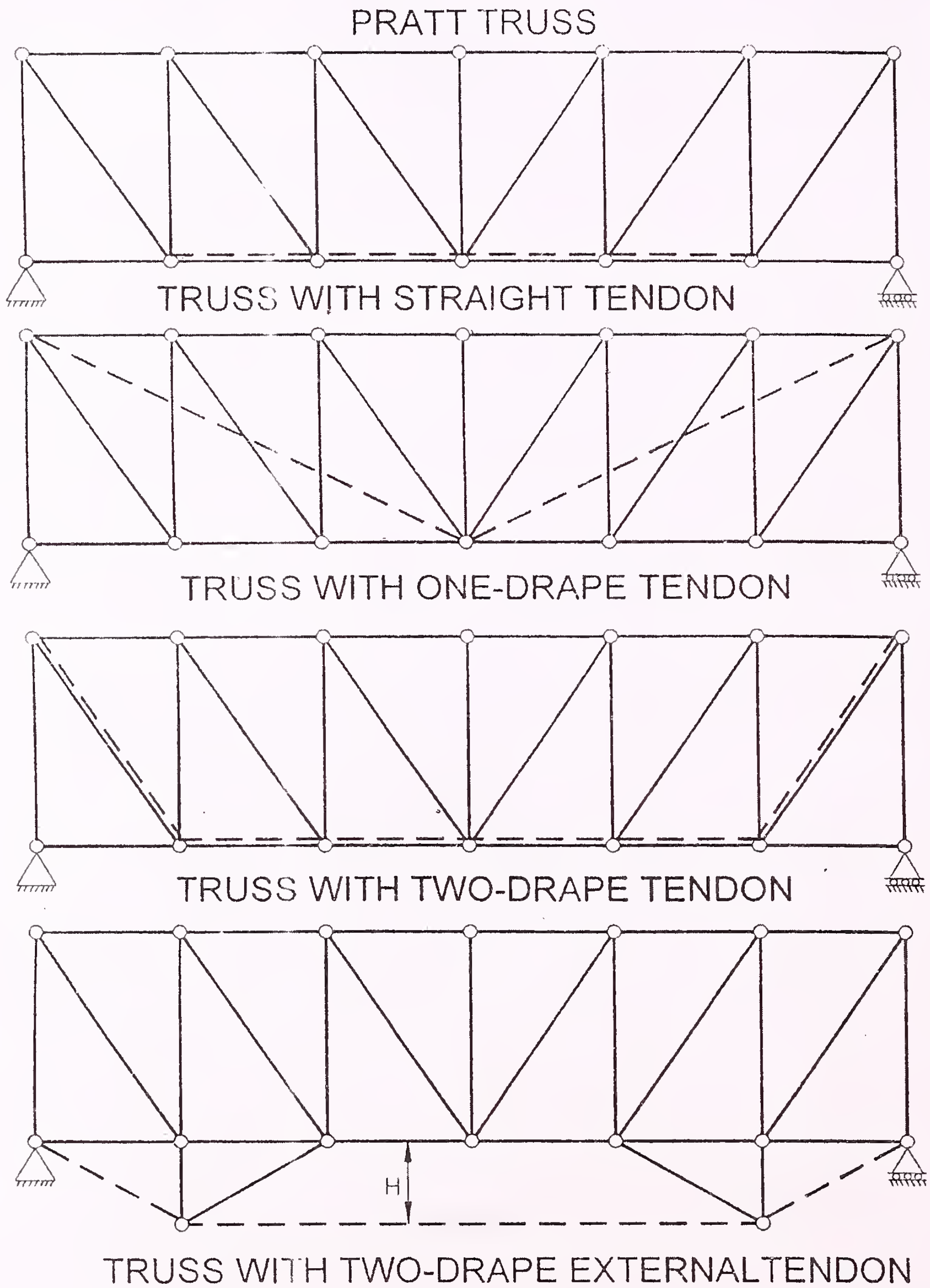


Fig. – 5 Pratt Truss Showing Different Single Tendon Configurations

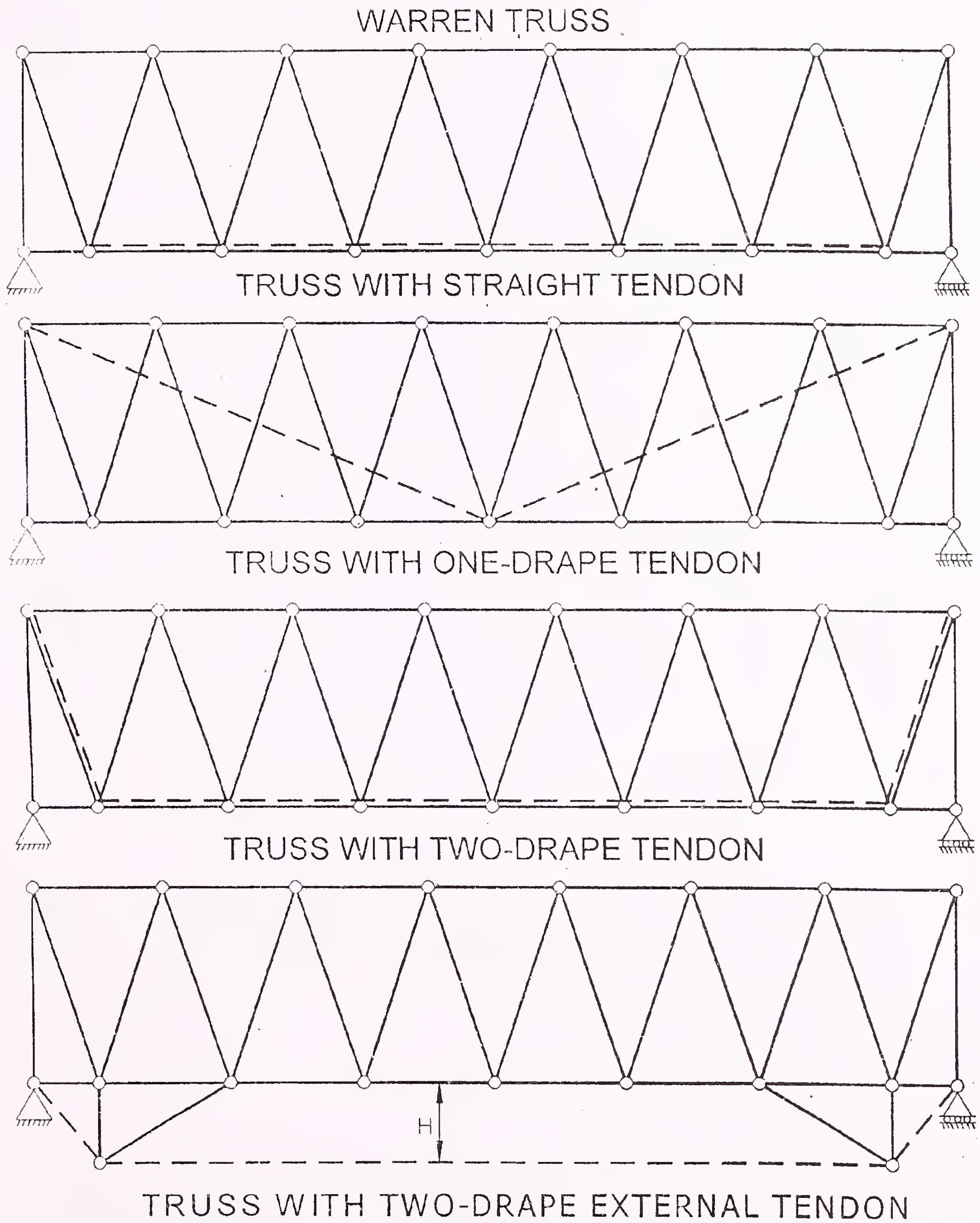


Fig. – 6 Warren Truss Showing Different Single Tendon Configurations

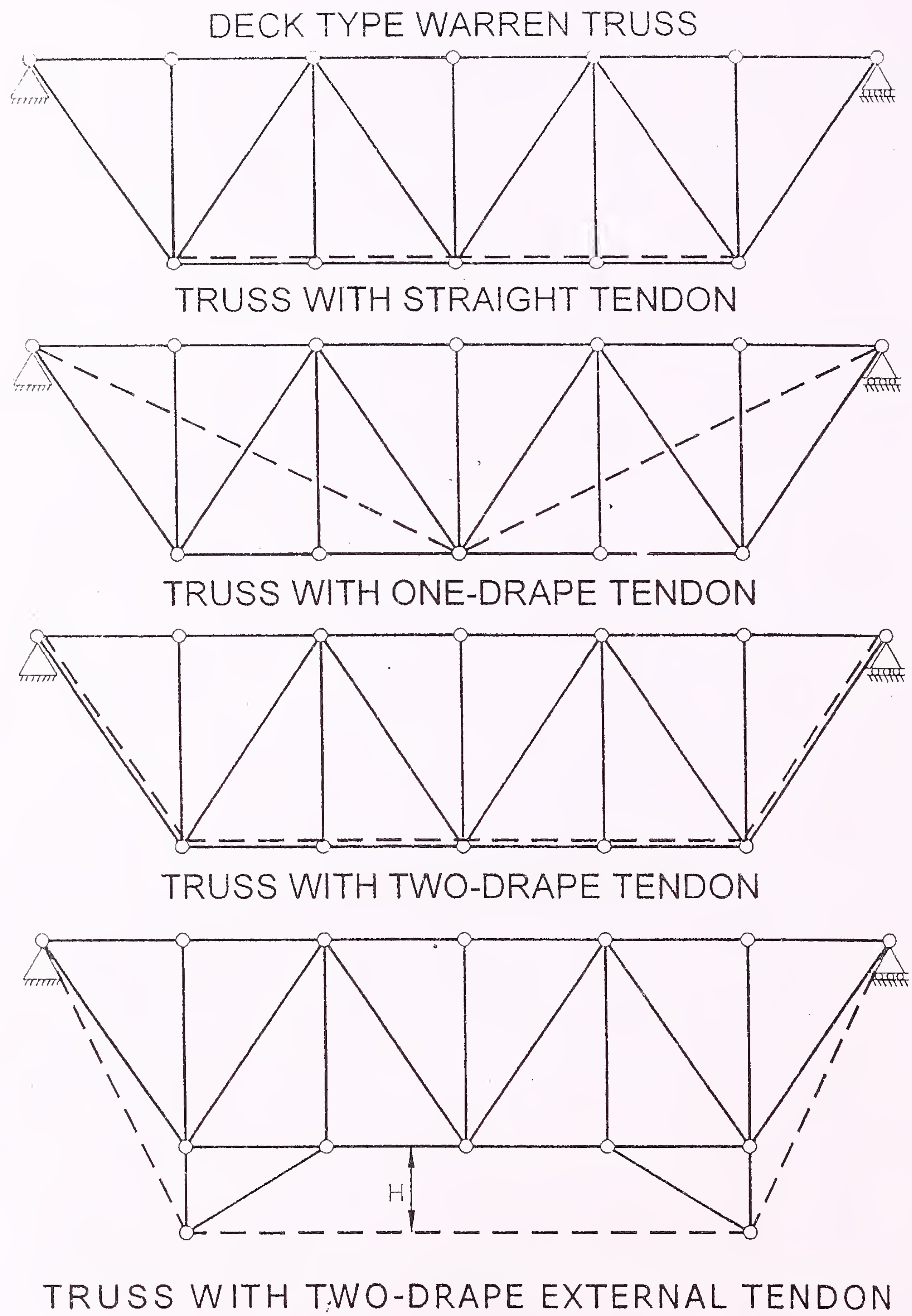


Fig. – 7 Deck Type Underslung Warren Truss Showing Different Single Tendon Configurations

7.3 Arches

Arches can be be prestressed in various ways as shown below.

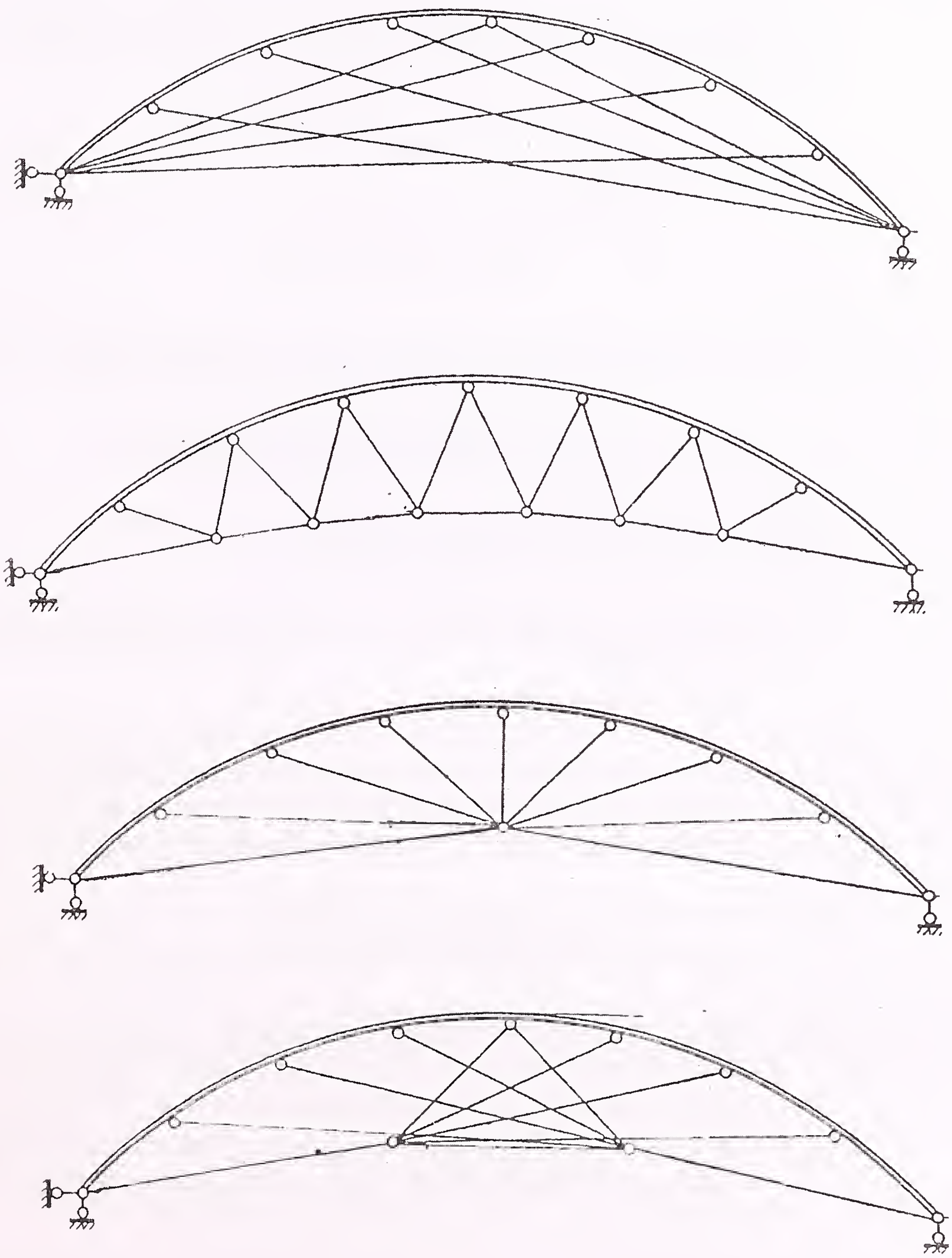


Fig. – 8 Arches prestressed by tendons

8. DIFFERENT METHODS FOR PRESTRESSING STEEL STRUCTURES USING TENDONS

8.1 Stressing separate structural units like beams, trusses and arches etc.

8.2 Tensioning of guys in suspended/combined system to increase the rigidity/capacity to take compressive loads.

8.3 Creating forced elastic deformation in some of the component parts to cause internal stresses in the units, usually opposite to that caused by external loads.

8.4 Inserting a tensioned high strength wire in the rolled sections, later being prestressed.

9. LOADS AND FORCES

In addition to loads, forces and load combinations as per IRC:6-2000 the effect of prestressing at different stages shall be taken into account.

10. GENERAL DESIGN REQUIREMENTS

10.1 **Rolled Beams/Plate girders** (Approximate spans upto 15.0 m for rolled beams, 40.0 m for plate girders).

(i) Profile of tendon shall be considered as straight/curved as the case may be.

(ii) Tendon shall be placed below centroid of girder (inside) or below the bottom chord under tension (outside). Elevation of tendon at mid span may be 0.15 h (maximum) from bottom of plate girder, where h = Depth of Beams/Plate girders.

(iii) For better utilization of section, the ratio $a=h_2/h_1$ shall preferably be between 1.5 to 2.0 for all asymmetric sections

(iv) Parameter characterizing flexibility of web $k=h/t_w$ preferably shall be kept less than or equal to 85 for rolled beams and plate girders without vertical stiffeners, $85 < k \leq 170$ for plate girders with vertical stiffeners and $170 < k \leq 240$ for plate girders with vertical & horizontal stiffeners & as appropriate as per IRC:24-2001.

(v) Parameter characterizing the distribution of material in the cross section shall equal to $m=A_w/A$ is 0.55 to 0.60.

(vi) Area of tendon

$$A_t = \frac{X + \Delta X}{f_t}$$

Where A = area of cross section of beam,
A_t = area of tendon,
A_w = area of web,
h = depth of web $\approx h_1 + h_2$,
tw = thickness of web,
h_1 & h_2 = top & bottom fibre distances,
X = Prestressing force,
Δ X = Self stressing force,
f_t = allowable stress in tendon.

For general theory and sample problems refer Appendices 1 – 3.

10.2 Trusses/Arches

(i) Profile of tendon shall be considered as straight/bent.

(ii) The tendon may be placed within the bottom chord of main truss (inside), or H below center line of bottom chord of main truss (outside) as shown in Figs.5, 6 & 7.

11. MAXIMUM POSSIBLE PRESTRESSING FORCE

Maximum value of possible prestressing force P shall be calculated on the basis of general profile of the tendon(s) in beams & girders of trusses/arches.

12. SELF STRESSING FORCE

With the application of Vehicular/pedestrian load, prestressing force increases with load applied (known as self stressing force). The increase in force in tendons due to further elongation or shortening of tendons at different stages during loading shall be accounted for.

13. DEFLECTION

Reverse deflection due to prestressing shall be calculated with the aid of strength of materials formula. The net deflection (Y_L - Y_P) shall not exceed 1/600 of the span as in IRC 24:2001.

Where Y_L = Total deflection due to dead load imposed load and impact,

Y_P = Total upward deflection due to prestressing.

Deflection due to live load and impact should not exceed 1/800 of span.

14. BASIC PERMISSIBLE STRESSES

The basic permissible stresses for steel work shall be as per IRC:24-2001 clause 506.4.1.

15. COMBINED STRESSES

15.1 In addition to the usual load combinations (as per IRC:6-2000) the following load combinations shall also be checked :

(i) Dead Load + Prestressing Force

(ii) Total Load + Prestressing Force

Due cognisance of self stressing force shall be taken in the combinations.

15.2 Prestressed steel members subjected to both axial and bending stresses shall be checked for combined stresses as per clause 506.4.2 of IRC:24-2001. The combination of stresses shall not exceed the permissible limits for following :

(i) Axial stress + bending stress

(ii) Shear stress + bending stress

16. LATERAL STABILITY

A prestressed steel member behaves like a beam-column and shall be treated as a column loaded by an eccentric axial force. Column stability of the member and lateral torsional buckling of the section shall be checked between points of lateral support. Stability shall be ensured at each stage of loading by providing sufficient bracing to the member considered as a whole, as well as to the compression flange in particular.

17. SECONDARY STRESSES

In addition to the secondary stresses considered in clause 506.8.2 of IRC:24-2001, the secondary stresses developed due to prestressing in member shall also be taken into account.

18. LOSSES IN PRESTRESS

While assessing the stress in tendon during tensioning operation and later in service, due regard shall be paid to all the losses and variation of stress resulting from relaxation of tendon, friction, slip, anchorage and elastic shortening/elongation of member between anchorages during loading.

18.1 Loss of prestress due to relaxation of tendon shall be considered as per Clause 11.4 of IRC:18-2000.

18.2 Loss of prestress due to friction shall be considered only in case of curved/bent tendons at the point of mechanical guide provided to maintain the inclination and change of direction. Suitable mechanical diverters/pulleys/guides are to be fixed to the structure to provide smooth curves at bends of tendons.

18.3 For angular change in profile, the loss, which depends on the configuration, type, and material of the cable, shall be taken into account.

19. FATIGUE

In addition to standard requirements of fatigue checking and fatigue detailing applicable to steel bridges as per IRC:6 , IRC:24 & IS:800, special attention shall be paid to stress raisers in tension zones such as corners, sharp notches, abrupt changes in thickness of flange/web plates, anchorage locations, web adjacent to deviators where the prestressing cables do not extend the complete span length to bearings, and other such locations of stress concentration.

Cables may be prone to fretting fatigue at deviator locations. In general, cables are prone to fatigue over time due to the continuous variation in self stressing force with passage of vehicles.

20. PRESTRESSING EQUIPMENT

The types of prestressing equipment for steel bridges is similar to those used for concrete bridges. Reference is invited to IS:1343-1980.

21. ANCHORAGE

Requirements for anchorages are in keeping with Clause 12.1.4 of IS:1343-1980.

22. PROCEDURE FOR TENSIONING AND TRANSFER

Procedure for tensioning and transfer shall conform to the requirements of Clause 12.2 of IS : 1343-1980.

23. MEASUREMENT OF PRESTRESSING FORCE

The measurement of prestressing force shall be in keeping with Clause 12.2.2 of IS : 1343-1980.

24. ASSEMBLY OF PRESTRESSING STEEL

Assembly of prestressing steel shall be in conformance with Clause 11 of IS:1343-1980.

25. PROTECTION OF TENDONS

Adequate protection shall be provided to tendons against corrosion by cement grouting of duct made of medium/heavy duty GI pipe/High density polyethylene (HDPE) pipes meeting all requirements of IRC:18-2000.

26. PERIODIC INSPECTION

Periodic inspection of the bridge shall be carried out as per stipulations of IRC:SP:18 and IRC:SP:35. In addition, superstructure tendons shall be checked every two years by checking the prestressing force, corrosion protection of tendon, and anchorage.

27. LIST OF REFERENCES

1. Moukhanov, K., "Metal Structures". Mir Publishers, Moscow
2. Belenya, E., "Prestressed Load Bearing Metal Structures". Mir Publishers, Moscow.
3. Troitsky, M.S. " Prestressed Steel Bridges Theory & Design", Van Nostrand Reinhold, New York 1990. (This text alone gives numerous refrences)

28. APPENDIX

Refer Annexure 1 and 2 for derivation of formulae.

Refer Annexure 3 for Important formulae for prestressing & Numerical Examples 1 to 4

A1.1 ANALYSIS AND DESIGN OF A PRESTRESSED STEEL GIRDER

(Notations used are different from those in text)

The introduction of a tendon transfers a girder to a statically indeterminate system. When the tendon is located on the side of those girder fibers in tension balanced by compressive girder stresses, they provide an additional moment of internal forces.

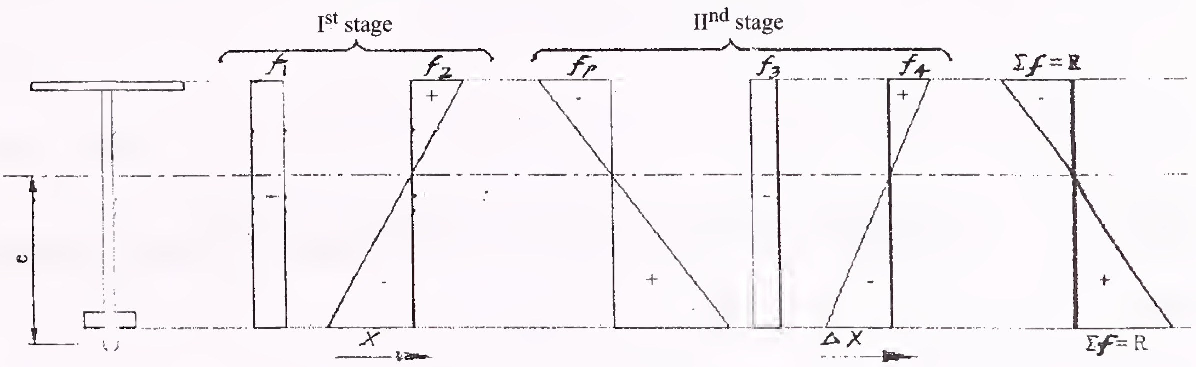


Fig. A1.1.01

The behavior of a girder in the elastic range, considering its cross section, may be divided into two stages (Fig. A1.1.01). In the first stage, a prestressing force X creates stresses

$$f_1 = -\frac{X}{A} \qquad \text{and } f_2 = \pm\frac{Xey}{I} = \pm\frac{M_P}{S} \tag{A1.01}$$

across the girder where M_P is the moment due to prestress X. In the second stage, the external load is applied until the stresses in the upper and bottom edges attain a yield point. At this point, the tendon is under an increment of prestressing force ΔX, which induces across the girder the additional stresses

$$f_3 = -\frac{\Delta X}{A} \qquad \text{and } f_4 = \pm\frac{\Delta Xey}{I} \tag{A1.02}$$

of an opposite sign to those stresses under external loading.

$$f_P = \frac{M\,y}{I} \tag{A1.03}$$

The resulting stresses for the compression edge are

$$f_c = -\frac{M}{S_1} - \frac{X + \Delta X}{A} + \frac{(X + \Delta X)e}{S_1} < F \tag{A1.04}$$

and for an edge in tension

$$f_t = \frac{M}{S_2} - \frac{X + \Delta X}{A} - \frac{(X + \Delta X)e}{S_2} < F \tag{A1.05}$$

and for a tendon

$$f_{td} = \frac{X + \Delta X}{A_t} < F_t \tag{A1.06}$$

where

X = a prestressing force
ΔX = an increment of tendon force
M = a bending moment due to external load ($DL+LL+SIDL$)
S_1 = a sectional modulus for a compressed edge
S_2 = a sectional modulus for a tensioned edge
A = a cross-sectional area of a girder
A_t = a cross-sectional area of a tendon

e = eccentricity of a tendon with respect to the centroid of a girder cross section
f_c = a compressive stress
f_t = a tensile stress
F_t = a permissible stress of tendon material
F = an allowable stress of girder material

It should be noted here that a girder may lose its load carrying capacity under prestressing if either the strength of the compressed flange is inadequate or

$$f_c = -\frac{X}{A} - \frac{Xe}{S_2} > F \qquad \text{(A1.07)}$$

and if the local stability condition is not met, which will be investigated in the section treating buckling.

A1.2 OPTIMUM PARAMETERS OF AN ASYMMETRICAL GIRDER

A1.2.1 Geometric Cross-Sectional Characteristics

For the determination of the optimum geometric parameters of a girder, it is convenient to express them in dimensionless parameters. In the case of a single-span girder having an asymmetric cross section prestressed by a straight tendon located at the bottom chord of a mixed span section, the main geometric cross-sectional characteristics are shown in Fig. A1.2.01.

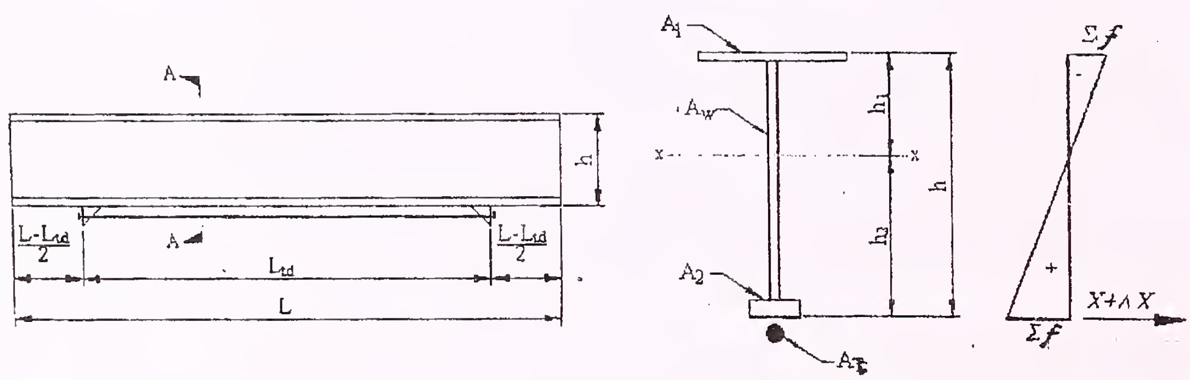


Fig. A1.2.01 Determination of the optimum parameters of a girder

Assume that the centers of gravity of the tendon's cross-sectional area A_t and the girder bottom chord are located at the same level and that the web height is equal to that of the girder $h_w = h = h_1 + h_2$. The following dimensionless parameters are thus introduced:

1. *Parameter characterizing the asymmetry of an I girder :*

$$a = \frac{h_2}{h_1} = \frac{S_1}{S_2} \qquad \text{(A1.08)}$$

2. *Parameter Characterizing the flexibility of a web :*

$$k = \frac{h}{t_w} \qquad \text{(A1.09)}$$

3. *Parameter characterizing the distribution of material in a section :*

$$\kappa = m = \frac{A_w}{A} \qquad \text{(A1.10)}$$

where $A = A_1 + A_2 + A_w$ = the total cross-sectional area of the I-Girder.

4. *Cross-sectional area of the flanges :* Taking the first moment, with respect to the axis at the bottom flange

$$h_2 A = A_1 h + Am\left(\frac{h_1 + h_2}{2}\right) \quad \text{(A1.11)}$$

and introducing a ratio $a = h_2/h_1$, we obtain

$$A_1 = A\left(\frac{a}{a+1} - \frac{m}{2}\right) \quad \text{(A1.12)}$$

and similarly

$$A_2 = A\left(\frac{1}{a+1} - \frac{m}{2}\right) \quad \text{(A1.13)}$$

5. *Heights h, h_1, and h_2:* From basic parameters, we have

$$A_w = Am = ht_w = \frac{h^2}{k} \quad \text{(A1.14)}$$

and

$$h = \sqrt{Akm} \quad \text{(A1.15)}$$

From

$$h = \sqrt{Akm} = h_1 + h_2 = h_1 + ah_1 \quad \text{(A1.16)}$$

Hence

$$h_1 = \frac{\sqrt{Akm}}{1+a} \quad \text{(A1.17)}$$

and

$$h_2 = h - h_1 = \frac{a\sqrt{Akm}}{1+a} \quad \text{(A1.18)}$$

6. *Moment of inertia I_x with respect to the axis x-x:*

$$I_x = \frac{1}{12} t_w h^3 + t_w h\left(h_2 - \frac{h}{2}\right)^2 + A_1 h_1^2 + A_2 h_2^2 \quad \text{(A1.19)}$$

After substituting the corresponding values from expressions (A1.08), (A1.09), (A1.10), (A1.11), (A1.12), (A1.13), we obtain

$$I_x = A^2 km \frac{6a - (a+1)^2 m}{6(a+1)^2} \quad \text{(A1.20)}$$

7. *Section moduli S_1 and S_2 :*

$$S_1 = \frac{I_x}{h_1} = \sqrt{A^3 km}\, \frac{6a - (a+1)^2 m}{6(a+1)} \quad \text{(A1.21)}$$

and

$$S_2 = \frac{I_x}{h_2} = \sqrt{A^3 km}\, \frac{6a - (a+1)^2 m}{6a(a+1)} \quad \text{(A1.22)}$$

8. *Section Modulus for a symmetric section :*

$$S = \frac{2I}{h} = \frac{\sqrt{A^3}\sqrt{n\kappa}(3 - 2\kappa)}{6} \quad \text{(A1.22a)}$$

where $n = \frac{h}{d}$, h and d are the height and thickness of the web in a symmetric section.

A1.3 DETERMINATION OF A BENDING MOMENT

Considering those parameters for an optimum girder cross section, which may take a maximum bending moment, the following equations may be used:

$$f_t = -\frac{X+\Delta X}{A} - \frac{M}{S_1} + \frac{(X+\Delta X)h_2}{S_1} = F \tag{A1.23}$$

$$f_b = -\frac{X+\Delta X}{A} + \frac{M}{S_2} - \frac{(X+\Delta X)h_2}{S_2} = F \tag{A1.24}$$

$$f_{bf} = \frac{X}{A} + \frac{Xh_2}{S_2} = F \tag{A1.25}$$

Equations (A1.23) and (A1.24) indicate that from a bending moment M due to an external load, it follows that the force due to prestressing X and increment of stress at the top edge and bottom tendon are equal to an allowable stress F.

By introducing into equations (A1.23) through (A1.25) a factor β equal to

$$\beta = \frac{X+\Delta X}{X}$$

we obtain

$$-\frac{M}{S_1} - \frac{\beta X}{A} + \frac{\beta X h_2}{S_1} = F \tag{A1.26}$$

$$\frac{M}{S_2} - \frac{\beta X}{A} - \frac{\beta X h_2}{S_2} = F \tag{A1.27}$$

$$\frac{X}{A} + \frac{Xh_2}{S_2} = F \tag{A1.28}$$

We solve equations (A1.26) and (A1.27) with respect to the design bending moment M after eliminating in them the value X by using equation (A1.28) and expressing their geometric characteristics through dimensionless parameters with the aid of formulas (A1.09) to (A1.20) we have

$$M = \frac{FA\sqrt{Akm}}{6}\cdot\frac{\left[6a-(a+1)^2 m)\right]\left[6a-(a+1)(1-\beta)m\right]}{(a+1)\left[6a-(a+1)m\right]} \tag{A1.29}$$

and from equation (A1.27)

$$M = \frac{FA\sqrt{Akm}}{6}\cdot\frac{\left[6a-(a+1)^2 m\right](1+\beta)}{a(a+1)} \tag{A1.30}$$

Equalizing expressions (A1.29) and (A1.30) we obtain an equation containing the parameters a, m, β for the optimum stressed state

$$\frac{6a-(1-\beta)(a+1)m}{6a-(a+1)m} = \frac{1+\beta}{a} \tag{A1.31}$$

Hence, the values of parameters m and a are

$$m = \frac{6a\left[a-(1+\beta)\right]}{(a+1)\left[a(1-\beta)-(1+\beta)\right]} \tag{A1.32}$$

and parameter a in terms of m and β is

$$a = \frac{m\beta - 3(1+\beta) - \sqrt{m^2 \quad 6m(1+\beta)^2 + 9(1+\beta)^2}}{m(1-\beta)-6} \tag{A1.33}$$

After substituting m from equation (A1.32) into equation (A1.33), we obtain the expression for the bending moment

$$M = fC\sqrt{A^3 k} \tag{A1.34}$$

and

$$A = \sqrt[3]{\frac{M^2}{C^2 f^2 k}} \tag{A1.35}$$

where

$$C = (1+\beta)\sqrt{\frac{6a^3(1-a)^2[a-(1+\beta)]}{(a+1)^3[a(1-\beta)-(1+\beta)]^3}} \tag{A1.36}$$

An analysis shows that by assuming $m = 0.55$ (for $\beta = 1$), the value of parameter C for all the values of a and β remains practically constant.

The above assumptions produce an error in the maximum value of the bending moment of less than 1%. If the assumed value of m is satisfied in equation (A1.33), and the expression is obtained for the optimum asymmetry of the cross section a and accordingly for C as a function of parameter β

$$a_{opt} = \frac{3 + 2.45\beta + \sqrt{0.303 + 5.7(1+\beta)^2}}{5.45 - 0.55\beta} \tag{A1.37}$$

Therefore, if the coefficient β is known, it is possible, after the value of C is obtained and the flexibility of the web k is specified, to calculate with the aid of equation (A1.35) the required area A of the girder cross section. Also, after the value of the optimum asymmetry a of the cross section is found from equations (A1.33), all other girder parameters can thus be determined from equations (A1.09) and (A1.19).

A1.4 EFFECT OF VARIOUS LOADINGS ON OPTIMUM PARAMETERS

We now consider three types of girder loadings as follows: the moments acting at supports, an uniformly distributed load, and a concentrated load at mid-span (Fig. A1 4.01),

The general expression for a prestressing force is obtained from equation (A1.25) after substituting values from equations (A1.09) to (A1.18).

$$X = \frac{FA[6a-(a+1)^2 m]}{(a+1)[6a-(a+1)m]} \tag{A1.38}$$

The value of an increment of prestressing force is

$$\Delta X = -\frac{\Delta_{11}}{\delta_{11}} = -\frac{\int \frac{M_1 M}{EI_x} dx}{\int \frac{M_1^2 dx}{EI_x} + \frac{l_t}{E_t A_t} + \frac{l_t}{EA}} \tag{A1.39}$$

For the types of loadings and girders having straight tendons at their bottom levels (Figure A1.4.01), the expression for ΔX may be simplified as follows

$$\Delta X = \frac{\frac{M_1}{I_x} A_m}{\left[\frac{M_1^2}{I_x} + \frac{l}{lA_t} + \frac{l}{A}\right] l_t} \tag{A1.40}$$

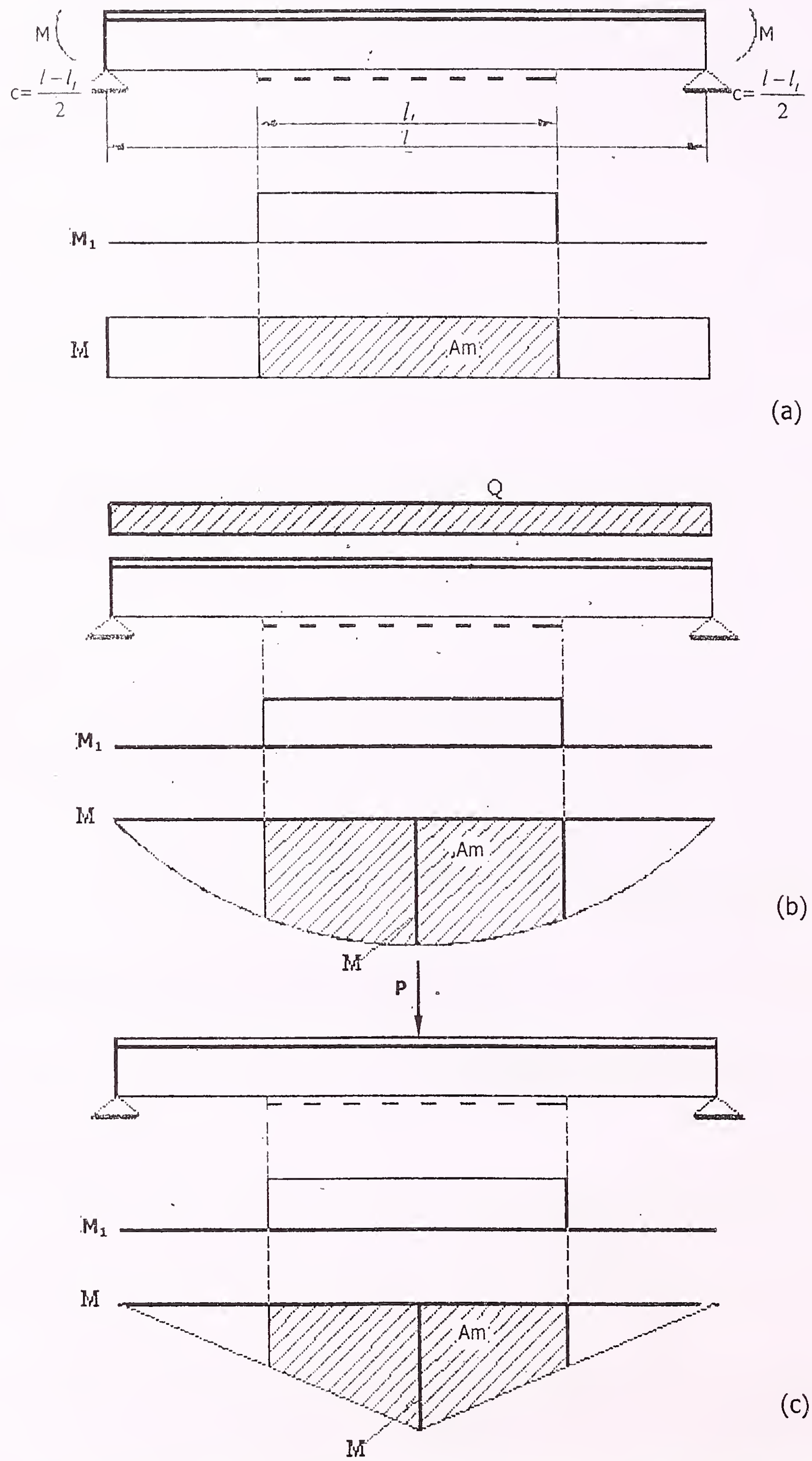


Fig. : A1.4.01 The determination of tendon forces for various types loading

where

A_m = an area of the diagram of a bending moment acting along the length of the tendon, $e = E_t / E$

$M_1 \quad = 1 \times h_2 =$ a bending moment due to $X = 1$ (unit force in tendon).

The cross sectional area A_t of a tendon may be found by considering the equilibrium of the diagram of stresses in a girder due to the action of its full design load (Fig. A1.2.01).

Projecting all the forces upon a horizontal axis, we obtain

$$(X + \Delta X) = A_t F_t = (A_1 - A_2)F \qquad \text{(A1.41)}$$

from which we get

$$A_t = \frac{F}{F_t}(A_1 - A_2) \qquad \text{(A1.42)}$$

Substituting the values A_1 and A_2 from equations (A1.12) and (A1.13), we have

$$A_t = A \frac{F}{F_t}\left(\frac{a-1}{a+1}\right) \qquad \text{(A1.43)}$$

In the following, we determine all the parameters for the prestressed girders shown in Fig. A1.4.01.

Case 1–A prestressed girder under moments acting at their supports (Fig. A1.4.01 (a)): The area of a diagram of bending moment A_m has the value

$$A_m = Ml_t \qquad \text{(A1.44)}$$

In this case of pure bending, the length of the tendon should be equal to that of its whole span or $\quad l_t = l$

Case 2–A girder under a uniformly distributed load (Fig. A1.4.01 (b)): The area of a diagram of bending moment A_m has the value

$$A_m = \left(\frac{2M + S_2 F}{3}\right) l_t \qquad \text{(A1.45)}$$

where $M_1 = S_2 F =$ the bending moment value, which is taken only by the girder cross section, the prestressing of the later than being unnecessary.

Under an uniformly distributed load, the length of a tendon is determined by the conditions of total capacity of the girder cross section at the location of its anchorages

$$M_a = S_2 F = FA\sqrt{Akm}\,\frac{6a - (a+1)^2 m}{6a(a+1)} \qquad \text{(A1.46)}$$

$$M_a = \frac{4m}{l^2}\left(lc - c^2\right)$$

and $\quad c = \dfrac{1}{2}\left(l - \sqrt{l - \dfrac{M_a}{M}}\right) \qquad \text{(A1.47)}$

$$l_t = l - 2c = \sqrt{1 - \frac{M_a}{M}} \qquad \text{(A1.48)}$$

By substituting $M_a = S_2 F$ and using equation (A1.34), $M = FCA\sqrt{Ak}$, we obtain

$$l_t = l\sqrt{1 - \frac{\sqrt{m}}{C} \cdot \frac{6a - m(a+1)^2}{6a(a+1)}} = l\sqrt{\alpha} \qquad \text{(A1.49)}$$

Case 3–A girder under a concentrated load at mid-span (Fig. A1.4.01 (c): The span of a diagram of bending moment A_m has the value

$$A_m = \frac{(M + S_2 F)}{2} l_t \qquad \text{(A1.50)}$$

Under a concentrated load, the length of the tendon is this determined as follows:

$$M_a = \frac{2cM}{l} \qquad c = \frac{M_a}{M} \cdot \frac{l}{2}$$

By substituting $M = FCA\sqrt{Ak}$

$$c = \frac{l}{2} \cdot \frac{\sqrt{m}}{C} \cdot \frac{6a - (a+1)^2 m}{6a(a+1)} \tag{A1.51}$$

and $$l_t = l - 2c = l\left[1 - \frac{\sqrt{m}}{C} \cdot \frac{6a - (a+1)^2 m}{6a(a+1)}\right] = \alpha l \tag{A1.52}$$

Substituting into equation (A1.40) the value of A_m for respective loading patterns and expressing other parameters through the values A, a, k, m and μ, we obtain formulas that define increment prestressing forces for all loading cases.
The value of μ is expressed as follows:

$$\mu = \frac{E_t}{E} \cdot \frac{f}{f_t} \tag{A1.53}$$

The resulting value of ΔX is substituted into equations for the optimum stressed state, and having solved them for M, we obtain formulas for bending moments expressed in terms of A, a, k, m and μ.

Equating the magnitudes of the bending moments, we obtain expressions that set up the relationship between a and m and determine the optimum stress state of the beam in bending.

1. For pure bending

$$\frac{6a}{[6a-(a+1)m][\mu(a-1)+2]} = \frac{2}{a} \tag{A1.54}$$

2. For an uniformly distributed loading

$$\frac{6a-(a+1)^2 m}{(a-1)[6a-(a+1)m]} = \mu . \frac{m(a+1)(5a+7)+6a(A-7)}{18.4(2-\alpha)-6\mu(\alpha+1)} \tag{A1.55}$$

3. For a concentrated load at mid-span

$$\frac{6a-(a+1)^2 m}{(a-1)[6a-(a+1)m]} = \mu . \frac{m(a+1)(3a+5)+6a(a-5)}{12a(2-\alpha)-4\mu(a+1)} \tag{A1.56}$$

In the following we develop Table A1.4.01, showing three different cases of a loaded beam (Fig.A1.4.01 (a), (b), and (c)). In all the cases, assuming values for μ=0.1, 0.2, 0.3 and 0.4, we calculated the values of a and C as follows.

Stage 1: Using $\mu = 0.55$ and substituting into formula (A1.54)

$$\frac{6a}{[6a-(a+1)m][\mu(a-1)+2]} = \frac{2}{a}$$

we obtain the corresponding value of a.

Stage 2: Considering formula (A1.36)

$$C = (1+\beta)\sqrt{\frac{6a^3(1-a)^2[a-(1+\beta)]}{(a+1^3)[a(1-\beta)-(1+\beta)]^3}}$$

and from the condition obtain a maximum value of C

$$\frac{dC}{da} = 0$$

we obtain the relation showing β in function of value of a, as follows:

$$\frac{1}{a(a+1)} - \frac{1-\beta}{a(1-\beta)-(1-\beta)} + \frac{3a-2(1+\beta)-1}{3(a-1)[a-(1+\beta)]} = 0 \qquad \text{(A1.57)}$$

By substituting into this relation the known value of a, we obtain β.

Stage 3: After substituting into formula (A1.36) known values of a and β, we obtain C.

Table A1.4.01 shows, for each value of μ, the corresponding values of a and c.

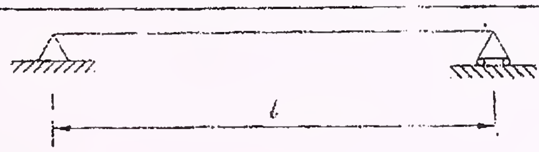
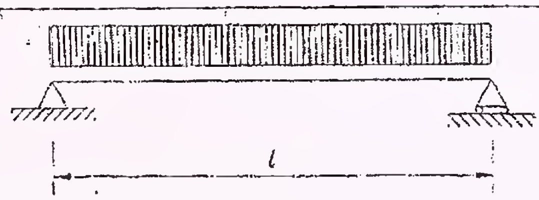
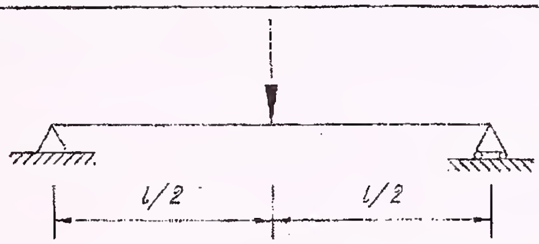
Type of loading	μ	a	C	Length of Tendon
	0.1	1.87	0.348	$l_t = l$
	0.2	2.11	0.369	
	0.3	2.56	0.399	
	0.4	3.60	0.446	
	0.1	1.83	0.344	$l_t = l\sqrt{\alpha}$
	0.2	1.98	0.357	
	0.3	2.16	0.371	
	0.4	2.36	0.384	
	0.1	1.82	0.342	$l_t = \alpha l$
	0.2	1.94	0.353	
	0.3	2.06	0.363	
	0.4	2.19	0.373	

Table A1.4.01: Variation of a and C with μ

A1.5 OPTIMUM DESIGN OF PLATE GIRDERS

Optimum design of prestressed plate girders may be summarized as follows:

1. The choice of girder cross section should be made considering its maximum carrying capacity under total loading.
2. The tensioning of the tendons should be maximum to realize the complete carrying capacity of the girder.
3. Considering the effective prestressing, the magnitude of the additional prestressing ΔX under live load is very important. Keeping the same cross section of girder will increase the capacity of the whole system.
4. The increase of ΔX depends on the tendon configuration and its length; ΔX increases with the straight and shorter tendons installed along bottom flange of the girder.
5. An increase in girder height and reduction of web thickness leads to the reduction of girder weight.

6. It is advantageous to design prestressed girder as an asymmetric section, using a larger top flange. This is because at the prestressed girder top flange participates more intensively and the resulting displacement of the neutral axis upward increases the eccentricity of the tendon.
7. The load carrying capacity and the rigidity of the structure may be increased by use of multistage prestressing, in which the prestressing and the loading of a structure are carried out in several steps.

A1.6 DEFLECTION

Prestressed girders are more prone to deformation in their elastic range of behavior than are conventional girders as they generally have a smaller sectional area and therefore a lesser moment of inertia. However, the positive characteristics of prestressing are as follows:

- The stiffness of the girders are increased
- Prestressed girders usually have substantially smaller deflections
- A girder may have a smaller construction depth, although its stiffness will be the same as that for a girder without prestressing

The design deflection of a prestressed girder in bending is calculated as follows:

$$\Delta = \Delta_{D+L} - \Delta_x - \Delta_{xl} \qquad \text{(A1.58)}$$

where Δ_{D+L} = a non-prestressed girder deflection under dead and live loads

Δ_x and $\Delta_{xl}^{\cdot}$ = the reverse deflection of a girder due to prestressing of the tendon

1. The deflection due to dead and live loads will be determined by conventional formulas from statics considering a non-prestressed girder.
2. When a tendon is placed at the bottom cross sectional area of a simple span girder and is prestressed, a deflection originates in an upward vertical deflection. A deflection due to dead and live loads is then produced in the opposite direction.

The deflection due to prestressing is calculated by applying the virtual work method using the general equation

$$\Delta = \int_0^l \frac{M_i m_k}{E_s I} dl + \int_0^l \frac{N_i n_k}{E_s A} dl \qquad \text{(A1.59)}$$

where M_i and N_i are the moments and axial loads, respectively, produced by the corresponding loads on the statically determinate structure. The term m_k represents the moment under the virtual unit force applied in the direction in which the deflection is sought. The second term in equation (A1.59) is applied when parabolics of polygonal tendons are used. However, it is usually neglected because its order of magnitude is relatively small.

The determination of a prestressed girder deflection is shown for a simple span girder, havin its tendon shorter than its span, in Fig. A1.6.01

The bending moment due to force X_t, as shown in fig. A1.6.01(a), is

$$M_a = -X_t e \qquad \text{(A1.60)}$$

where X_t = $X + \Delta X$, the total tendon force

X and ΔX = the prestressing and increment of prestressing force in a tendon, respectively

e = the eccentricity of a tendon force with respect to beam centroidal axis x-x

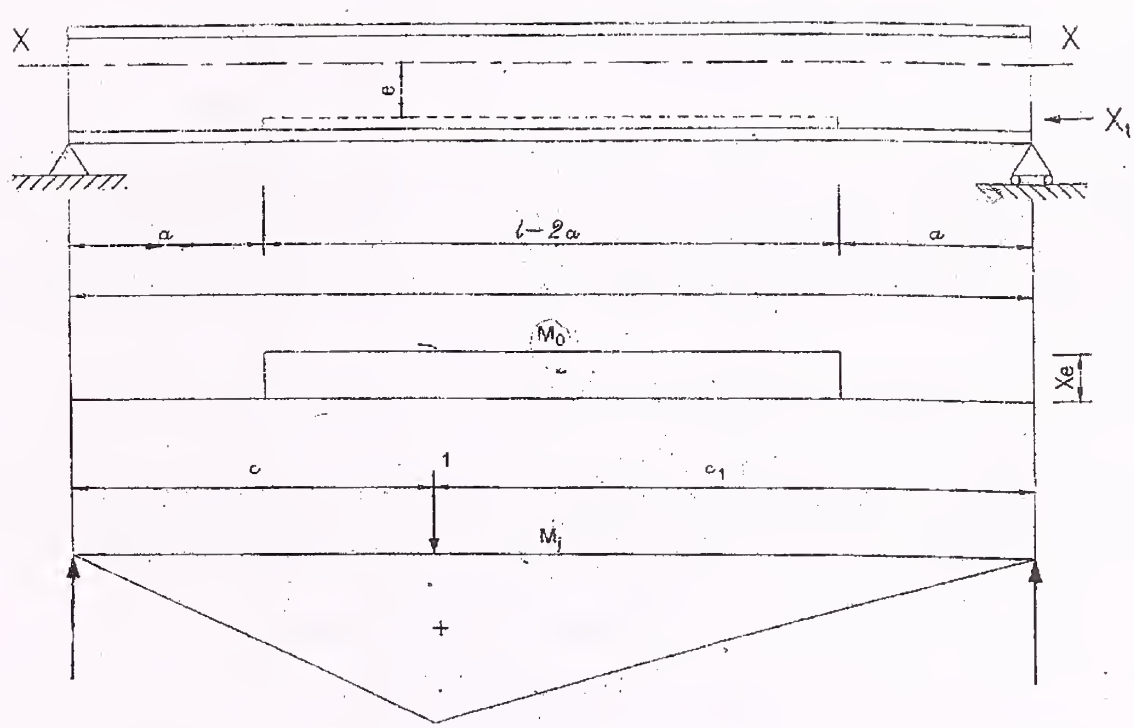


Fig. A1.6.01 The determination of tendon forces for various types loading

The deflection of a girder is expressed by the first term of equation (A1.59)

In fig. A1.6.01(a), the moment diagram for a given tendon's force is shown, and in fig. A1.6.01(b), the same is shown for a unit load at the mid-span of a girder, where the maximum deflection occurs.

For the case under consideration,

$$M_1 = \frac{c_1}{l}x \qquad (\text{ for } 0 \le x \le c\ ) \qquad \text{(A1.61)}$$

$$c_1 = (1-c)$$

where M_1 is a bending moment due to a unit force acting at the point and direction of the required displacement.

After introducing expressions (A1.60) and (A1.61) into equation (A1.59), we have

$$\Delta_{max} = -\frac{2}{EI}\int_{a}^{c}\frac{c_1}{l}X_1 xe dx = -\frac{X_1 ec_1}{EI}\left(c^2 - a^2\right) \qquad \text{(A1.63)}$$

$$= \frac{X_1 el^2}{8EI}\left[1 - 4\left(\frac{a}{l}\right)^2\right] \quad \text{at } c = c_1 = l/2.$$

The total deflection will be determined as the summary of those deflections for a non-prestressed loaded girder and a girder under the action of a prestressed tendon.

A1.7 BUCKLING STRENGTH OF PLATE GIRDERS

Stability of Plate Girder Bottom Chord during Prestressing

Analysis of the behavior of a steel member prestressed by a tendon installed along the center of gravity of the member and connected to it at separate points proved that the member loses its stability only between tendon connections. The tendons are usually connected to the girder at certain intervals by means of diaphragms, ribs, clamps and other types of grips which allow a longitudinal displacement of the tendon but prevent the girder from buckling during prestressing.

The safety may be calculated by checking the bottom chord for buckling using formula

$$\sigma_x \leq \varphi\sigma \tag{A1.64}$$

where φ is the coefficient of lateral bending or buckling determined based on the flexibility of the girder chord with respect to the vertical axis with the free length of the bottom chord equal to the spacing between points of connection of the tendon to the bottom chord.

Determination of the coefficient of lateral bending

According to the Euler formula the critical stress of buckling is

$$\sigma_{cr} = \frac{\pi^2 E}{\left(\frac{l}{r}\right)^2} = \frac{\pi^2 E}{\lambda^2} \tag{A1.64}$$

The ratio of critical stress to the allowable stress of given steel may be expressed as follows

$$\varphi = \frac{\sigma_{cr}}{\sigma_{all}} \text{ and } \sigma_{cr} = \varphi\sigma_{all} \tag{A1.65}$$

However, the Euler formula is valid only until stress σ_{cr} remains within the proportionality limit. For structural steel with a proportionality limit of 30×10^3 psi and E = 30×10^6 psi, we find from equation (A1.64)

$$\frac{l}{r} = \sqrt{1000\pi^2} = 100 \tag{A1.66}$$

Hence, for $l/r < 100$ Euler's formula is not valid.

In this case the values of coefficient φ are determined on the basis of empirical data given by the Navier formula

$$\varphi = \frac{1}{1 + 0.00008(l/r)^2} \tag{A1.67}$$

The diagram in Fig. A1.7.01 indicates the values of coefficient φ in function of slenderness ratio.

Determination of Critical Prestressing buckling force

Compressive stress in the bottom chord due to prestressing force in the tendon is

$$\sigma_x = \frac{X}{A} + \frac{Xe}{S} \tag{A1. 68}$$

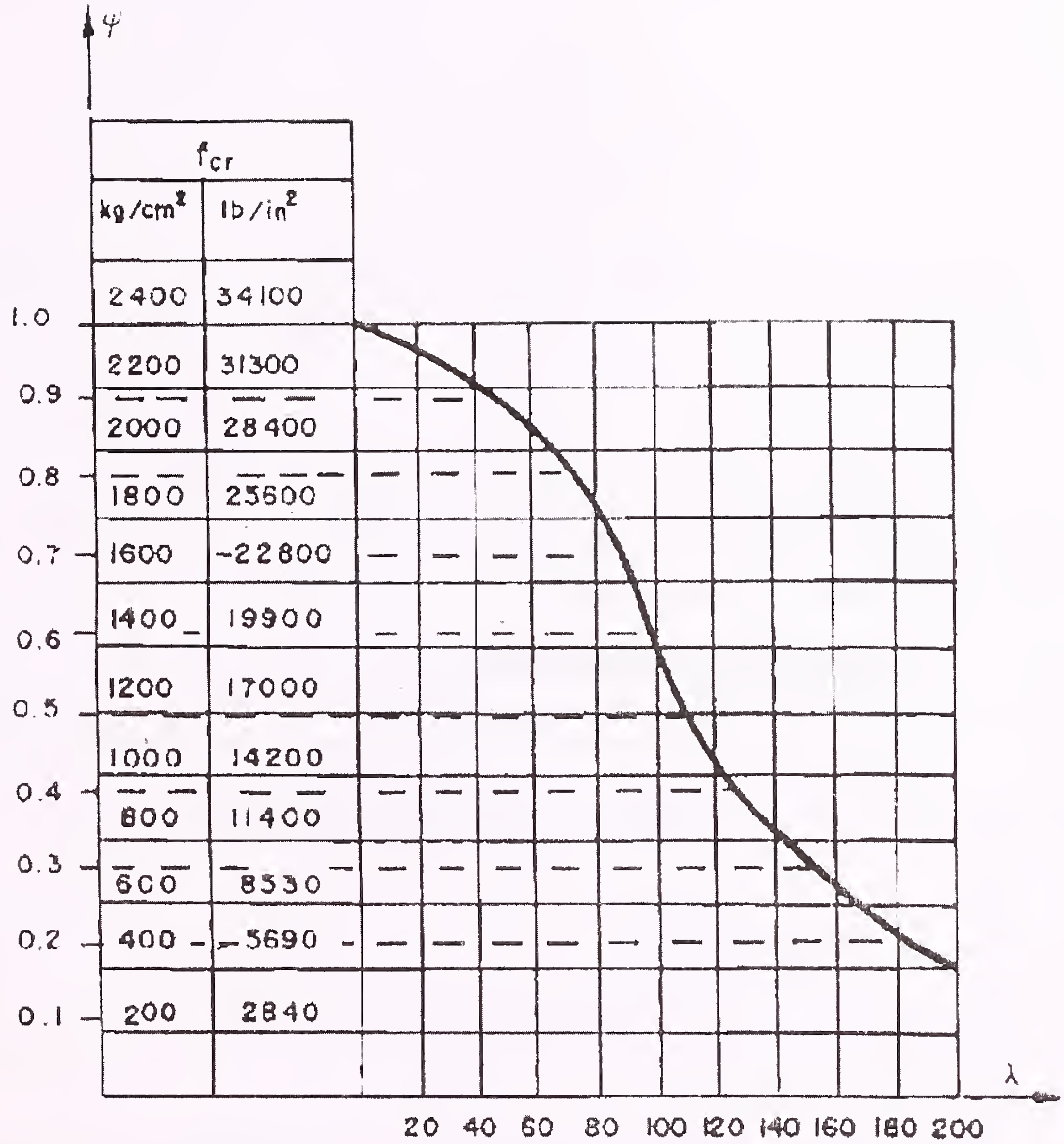


Fig. A1.7.01 Diagram of coefficient φ

Therefore, a minimum possible tendon force is, after substitution of (A1.68) into (A1.64)

$$\sigma\varphi = \frac{X(S + Ae)}{AS} \text{ and } X = \frac{\varphi\sigma AS}{S + eA} \tag{A1.69}$$

Let us find the coefficient φ for a prestressed girder having tendon connected at spacings a, shown in Fig. A1.7.02.

$$I_y = \frac{hb^3}{12} \qquad A = bh$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{hb^3}{12bh}} = \frac{b}{\sqrt{12}}$$

slenderness ratio $\lambda_y = \dfrac{a}{r_y} = \dfrac{a\sqrt{12}}{b}$ (A1.70)

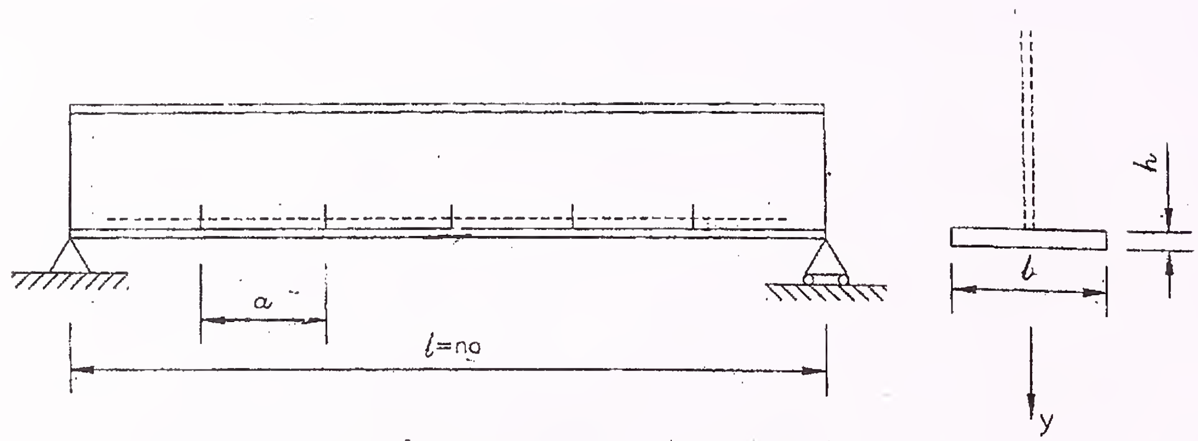


Fig. A1.7.02 Tendon connected to the bottom chord

Therefore, for given λ_y we may find the corresponding coefficient φ from Fig.A1.7.01 and calculate the permissible prestressing force X, from formula (A1.69)

A1.8 GIRDER UNDER LIVE LOAD

A1.8.1 Prestressing Due to Concentrated Load

The increment of the tendon force under concentrated load (Fig.A1.8.01) is

$$\Delta X = \frac{\int_a^{l-a} MM_1 e\,dx}{\int_a^{l-a} M_1^2 dx + \frac{I}{A} l_1 + \frac{EI}{E_t A_t} l_t}$$

$$M_1 = 1 \times e \qquad M_1^2 = \int_a^{l-a} e^2 dx = e^2 (1-2a) = e^2 l_t$$

The area of the moment diagram is

$$\int_a^{l-a} M dx = P\int_a^{x1} \frac{(l-1)x}{l} dx + P\int_a^{l-x1} \frac{x_1 x}{l} dx - p\int_0^{a} \frac{(l-x_1)}{l} x dx - p\int_0^{a} \frac{x_1 x}{l} dx = \frac{p}{2}\left(lx_1 - x_1^2 - a^2\right)$$

Therefore,

$$\Delta X = \frac{Pe\left(lx_1 - x_1^2 - a^2\right)}{2(1-2a)\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \qquad \text{(A1.71)}$$

A1.8.2 Girder under Truck Load

The absolute maximum bending moment due to action of truck load on a simple-span girder occurs when the middle of the span is halfway between the load closest to the resultant of all the loads on the span. The position of the truck to determine maximum moment is shown in Fig. A1. 8.01.

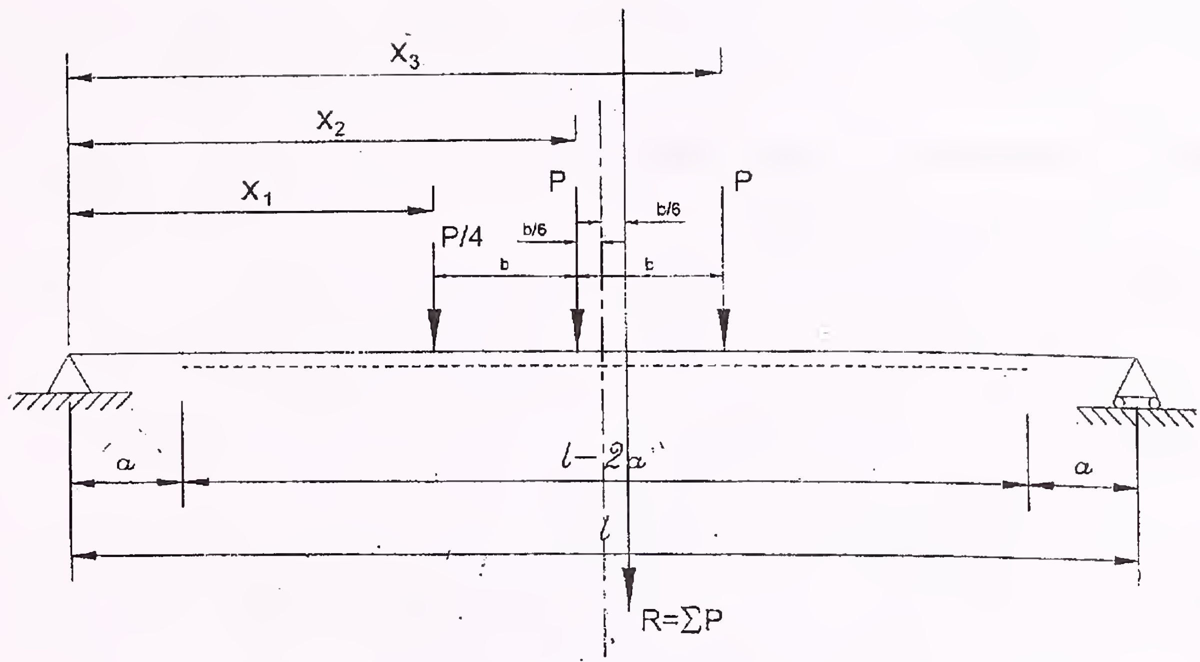


Fig. A1.8.01 Prestressed girder under single concentrated load.

In this case the intensity of the increment of prestressing force is calculated as summary for each concentrated load after formula (A1.71), or

$$\Delta X = \Delta X_1 + \Delta X_2 + \Delta X_3 \qquad \text{(A1.72)}$$

where, ΔX_1, ΔX_2 and ΔX_3 are componental increments under action of concentrated truck loads, respectively.

A1.8.3 Length of the Tendon in Girders Stressed by a Live Load

The length of the tendon in girders stressed by a live load is established as a function of the distance a from the bearing to the beginning of the tendon. This distance depends on the two requirements as follows.

The Maximum Distance, a_{max}

The maximum distance a_{max} is found from the condition that the cross section of the girder at the point of tendon anchoring is required to withstand the acting bending moment. At this point the girder cross section is considered without the tendon.

Before, a_{max} is attained, the strength of an asymmetric cross section is checked on the value of stresses in the bottom chord

$$M_a \le S_2 f \qquad \text{(A1.73)}$$

The admissible value of moment M_a may be calculated and the maximum value of a_{max} determined graphically, provided that the envelope of the diagram of moments are available, if the cross section as given by formula (A1.73) is known. If the cross section has not yet been selected, the value of the moment M_a at the anchor location may be found in terms of maximum design moment M as follows.

By substitution of the value from formula (A1.35)

$$A^3 = \frac{M^2}{C^2 f^2 k}$$

into the formula for section modulus (A1.22)

$$S_2 = \sqrt{A^3 km} \cdot \frac{6a - (a+1)^2 m}{6a(a+1)}$$

we have

$$S_2 = \frac{M}{Cf}\sqrt{m} \cdot \frac{6a - (a+1)^2 m}{6a(a+1)}$$

Using the value of C from formula (A1.36)

$$C = (1+\beta)\frac{a}{a+1}\sqrt{\frac{6a(1-a)^2[a-(1+\beta)]}{(a+1)[a(1-\beta)-(1+\beta)]^3}}$$

and substituting $\beta = 1$ and a = 1.71, we obtain C = 0.33, and for m = 0.55

$$M_a = S_2 f = \frac{M\sqrt{0.55}}{0.33} \cdot \frac{6\times 1.71 - 2.71^2 \times 0.55}{6\times 1.71\times 2.71} = 0.5M$$

Therefore M_a and a_{max} may be found, if the maximum design bending moment M in the girder and the envelope of moments are known.

By substitution into formula (A1.35)

$$A = \sqrt[3]{\frac{M^2}{C^2 f^2 k}}$$

the value of C form formula (A1.36) for $\beta = 1$, we obtain

$$A = \sqrt[3]{\frac{M^2}{f^2 k} \cdot \frac{a+1}{a^3\sqrt{3(2-a)(1-a)^2}}} \qquad \text{(A1.74)}$$

The Minimum Distance, a_{min}

The minimum distance a_{min} is found considering that the increment of stresses in the bottom chord at the end point of tendon should be tensile for any position of live load. The stresses are then checked for safe values.

If the tendon is anchored near the bearing, the live load may be positioned on the girder so as to produce a greater increment of stresses σ_x in the bottom chord of the girder due to the compression increment of the tendon than the increment of tensile stresses σ_l due to live load.

In this case the compressive stresses due to prestress X (generally equal to σ) add up with the resultant compressive stresses due to the action of external load, with the effect that the bottom chord is overstressed.

The closer the tendon to the bearing, the greater the probability of overstressing the bottom chord in the tendon anchoring zone where the bending moment from the external load will be small.

The initial equation for establishing the minimum distance from the girder support to the tendon anchoring is

$$\sigma_b' = \sigma_x = \sigma_t \geq 0 \tag{A1.75}$$

where σ_x = compressive stress due to increment of the compressive force in the tendon

σ_y = Tensile stress due to bending moment from external load in a section of girder without tension, calculated from the following formulas

$$\sigma_x = -\frac{\Delta Xe}{S_2} - \frac{\Delta X}{A} \text{ and } \sigma_t = \frac{M_t}{S_2} \tag{A1.76}$$

For a load located within the tendon length at a distance x from the support and a cross section located to the left of the load, we find the stress σ_b' from equation (A1.75) after substituting the following values.

From formula (A1.71)

$$\Delta X = \frac{Pe\left(lx_1 - x_1^2 - a^2\right)}{2(1-2a)\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)}$$

we designate by μ the expression

$$\mu = \frac{e}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \tag{A1.77}$$

and from (A1.76)

$$\sigma_x = -\Delta X\left(\frac{e}{S_2} + \frac{1}{A}\right) = -\Delta X\alpha \tag{A1.78}$$

where $\alpha = \frac{e}{S_2} + \frac{1}{A}$ (A1.79)

By designating the distance from the left support to the cross-section considered as η, we have

$$M_t = \frac{P(l-x)}{l}\eta \tag{A1.80}$$

By substituting into expressions (A1.75) the values from equations (A1.76), (A1.77), (A1.78), and (A1.79), we obtain

$$\sigma_b' = \frac{P\mu\alpha}{1-2a}\left(x^2 - xl - a^2\right) + \frac{P\eta(l-x)}{lS_2} \tag{A1.81}$$

Equating the derivative of σ_b' with respect to x to zero, we obtain the location of the load for which the compressive stresses in the cross section of coordinate η attain a maximum value.

$$\bar{x} = \frac{1}{2} + \frac{\eta}{2lS_2}\frac{(l-2a)}{\mu\alpha} \tag{A1.82}$$

We designate by γ the value

$$\gamma = \mu\alpha S_2 = \frac{e\left(e + \frac{S_2}{A}\right)}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \tag{A1.83}$$

where, from (A1.79)

$$\alpha S_2 = \left(e + \frac{S_2}{A}\right) \tag{A1.84}$$

and $$\bar{x} = \frac{1}{2} + \frac{\eta(l-2a)}{2\lambda l} \tag{A1.85}$$

To determine the distance a_{min} from the support, substitute $\bar{x}$ in formula (A1.81), assuming $\eta = a$, and equate σ_b' to zero.

By solving the resultant equation for a, we obtain

$$a_{min} = \gamma l$$

A1.8.4 Calculation of a Prestressed Girder under a Movable Concentrated Load

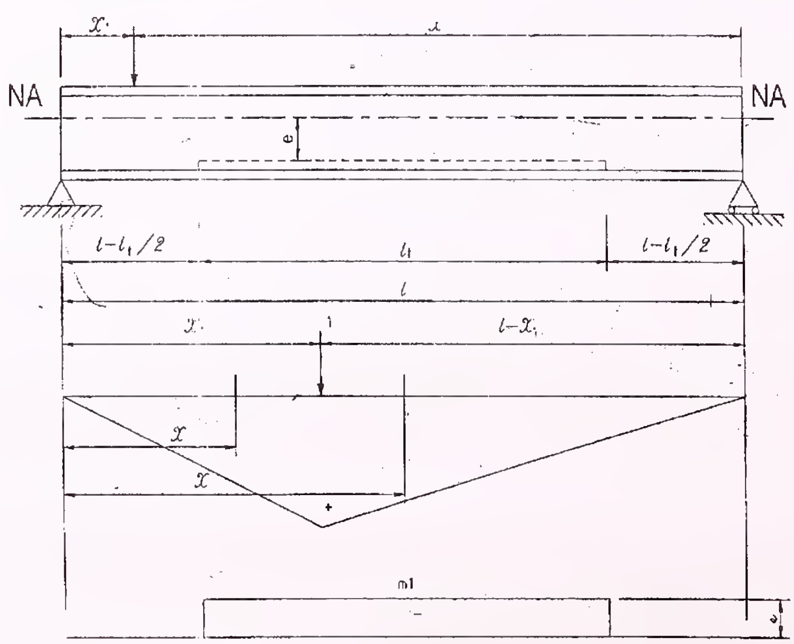


Fig. A1.8.02 Diagram of bending moments due to unit load.

1. We are considering a prestressed girder under the influence of a movable unit load (Fig. A1.8.02)

	In intervals
$M_x = \frac{x(l-x_1)}{l}$	$(0 \text{ to } x_1)$
$m_1 = -e$	$\left(\frac{l-l_t}{2} \text{ to } x_1\right)$
$M_x = \frac{x(l-x_1)}{l} - 1(x-x_1)$	$(x_1 \text{ to } l)$
$m_1 = -e$	$\left(x_1 \text{ to } \frac{l+l_t}{2}\right)$
$N_1 = 1$	$\left(\frac{l-l_t}{2} \text{ to } \frac{l+l_t}{2}\right)$

2. An increment of prestressing under an unit load acting along the tendon length is

$$\delta_{11}\Delta X_1 + \delta_{1L} = 0 \qquad \Delta X = -\frac{\delta_{1L}}{\delta_{11}}$$

$$\delta_{11} = \int_0^1 \frac{m_1^2}{EI}dx + \int_0^1 \frac{N_1^2 dx}{EA} + \int_0^1 \frac{N_1^2 dx}{E_t A_t}$$

But the integration over the interval 0 to $(l-l_t)/2$ and $(l+l_t)/2$ to 1 is zero; therefore in interval $(l-l_t)/2$ to $(l+l_t)/2$, we have

$$\delta_{11} = +\int_{l-l_t/2}^{l+l_t/2} \frac{1^2 dx}{EI} + \int_{l-l_t/2}^{l+l_t/2} \frac{dx}{EA} + \int_{l-l_t/2}^{l+l_t/2} \frac{dx}{E_t A_t} = \frac{l_t}{EI}\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right) \qquad \text{(A1.87)}$$

and

$$\delta_{1L} = \int \frac{m_1 M_x}{EI} dx$$

$$EI\delta_{1L} = \int_{l-l_t/2}^{x_1} \frac{-ex(1-x_1)dx}{1} + \int_{x_1}^{l+l_t/2} \frac{-ex_1(l-x)dx}{1} \qquad \text{(A1.88)}$$

$$= -\frac{e}{1}\left\{\left[\frac{x^2(l-x_1)}{2}\right]_{l-l_t/2}^{x_1} + \left[\frac{(1-x)^2 x_t}{2}\right]_{x_1}^{l+l_1/2}\right\}$$

$$= \frac{e}{2}\left[lx_1 - x_1^2 - \frac{(l-l_t)^2}{4}\right]$$

$$\Delta X_1 = \frac{e\left[lx_1 - x_1^2 - \frac{(l-l_t)^2}{4}\right]}{2l_t\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \qquad \text{(A1.89)}$$

This is the equation of ΔX for the unit load at x_1, which is actually the equation of the influence line of ΔX for an unit load acting along the length of the tendon.

3. For the case of a set of equal moving concentrated loads, ΔX will be the summation of all loads. In the particular case on n equal concentrated loads P, the incremental force ΔX will be equal to

$$\Delta X_2 = \sum \frac{Pe\left\{\left[lx_1 - x_1^2 - \left(\frac{(l-l_t)^2}{2}\right)\right] + \cdots \left[lx_n - x_n^2 - \left(\frac{l-l_t}{2}\right)^2\right]\right\}}{2l_t\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \qquad \text{(A1.90)}$$

or

$$\Delta X_3 = \sum \frac{Pe\left\{\left[l(x_1 + x_2 + \cdots + x_n) - (x_1^2 + x_2^2 + \cdots + x_n^2)\right] - n\left(\frac{l-l_t}{2}\right)^2\right\}}{2l_t\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \qquad \text{(A1.91)}$$

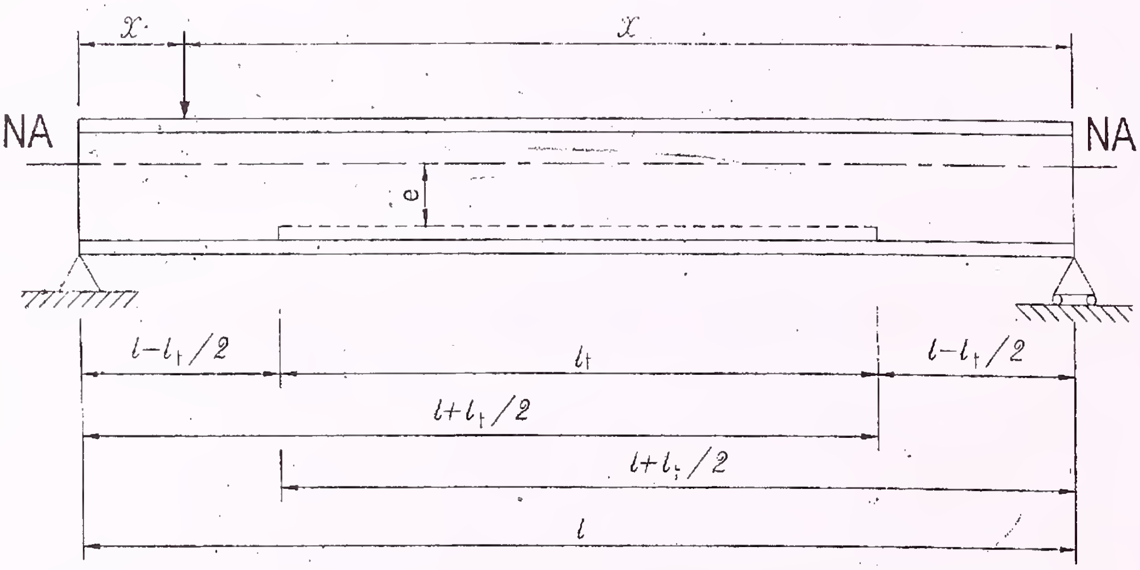


Fig. A1.8.03 Intervals along prestressed girders for action of unit load.

4. For the particular case when unit loading is acting before the point of a short tendon on the left side, $x_1 < (l - l_t)/2$ (Fig. A1.8.03)

$$M_x = \frac{x(l - x_1)}{l} \qquad \text{In intervals } (x_1 \text{ to } 1)$$

$$EI\delta_{1L} = \int_{l-l_t/2}^{l+l_t/2} \frac{ex_1(1-x)dx}{1} = -\frac{ex_1}{1}\left[\frac{(1-x)^2}{-2}\right]_{l-l_t/2}^{l+l_t/2} = -\frac{ex_1 l_t}{2} \qquad \text{(A1.92)}$$

and

$$\Delta X = \frac{ex_1 l_t}{2l_t\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} = \frac{ex_1}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \qquad \text{(A1.93)}$$

For a set of equal concentrated forces P

$$\Delta X_4 = \sum \frac{Pe(x_1 + x_2 + \cdots x_n)}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \tag{A1.94}$$

5. For the particular case when $x_t > l + l_t/2$

$$M_x' = \frac{x}{1}(l - x_1) \qquad (0 \text{ to } 1)$$

$$m_1 = -e \qquad \left(\frac{l - l_t}{2} \text{ to } \frac{l + l_t}{2}\right)$$

$$\delta_{1L} = \int_{l-l_t/2}^{l+l_t/2} \frac{-ex(l - x_1)}{1} dx$$

$$= -\frac{1(1 - x_1)}{1}\left(\frac{x^2}{2}\right)_{l-l_t/2}^{l+l_t/2} = -\frac{el_t(l - x_1)}{2} \tag{A1.95}$$

and

$$\Delta X_5 = \frac{Pel_t(1 - x_1)}{2l_t\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} = \frac{Pe(1 - x_1)}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \tag{A1.96}$$

For a set of concentrated loads acting be yond the right tendon anchorage

$$\Delta X_6 = \sum \frac{Pe[(l - x_1) + (l - x_2) + \cdots + (l - x_n)]}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)} \tag{A1.97}$$

6. Transformation of the denominator in expression (A1.91) can be found similar to previous calculation for δ_{11}, or after formula (A1.87)

$$\delta_{11} = \frac{l_t}{EI}\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)$$

For the calculation of δ_1, the maximum ordinate at the mid-span may be found from expression (A1.92), after the substitution $x_1 = 1/2$, namely,

$$\frac{1}{2}\left[lx_1 - x_1^2 - \frac{(l - l_t)^2}{4}\right] = \frac{1}{2}\left[\frac{l^2}{2} - \frac{l^2}{4} - \frac{(l - l_t)^2}{4}\right] = \frac{l_t(2l - l_t)}{8} \tag{A1.98}$$

The envelope curve to the maximum value of moment diagram due to moving loads is of a second-degree curve equation, having boundary conditions of zero at the two ends and the ordinate $[l_t(2l - l_t)]/8$ at mid-span. We assume such a parabolic equation expressed as

$$y = Ax^2 + Bx + C \tag{A1.99}$$

Applying boundary conditions

at $x = 0$ $\qquad y = 0$ and $\qquad C = 0$

at $x = \frac{1}{2}$ $\qquad y = \frac{l_t}{8}(2l - l_t)$ and $\frac{l_t}{8}(2l - l_t) = \frac{Al^2}{4} + \frac{Bl}{2}$

at $y = 0$ $\qquad Al + B = 0$

Solutions of the above equations yield

$$A = -\frac{l_t}{2l^2}(2l - l_t) \qquad B = +\frac{l_t l}{2l^2}(2l - l_t) \tag{A1.100}$$

After substitution of (A1.100) into (A1.99), we obtain

$$y = \frac{l_t(2l - l_t)}{2l^2}\left(-x^2 + lx\right) \tag{A1.101}$$

For a set of unit concentrated loads

$$y = \frac{l_t(2l - l_t)}{2l^2}\left[l(x_1 + x_2 + \cdots + x_n) - \left(x_1^2 + x_2^2 + \cdots + x_n^2\right)\right]$$

and considering that $\delta_{1L} = \sum(Mm/EI)$

$$EI\delta_{1L} = -\frac{el_t(2l - l_t)}{2l}\left[l(x_1 + x_2 + \cdots + x_n) - \left(x_1^2 + x_2^2 + \cdots + x_n^2\right)\right]$$

Considering that $\delta X = -(\delta_{1L}/\delta_{11})$, for a set of concentrated moving loads P, we have

$$\Delta X_6 = \sum \frac{Pe(2l - l_t)\left[l(x_1 + x_2 + \cdots + x_n) - \left(x_1^2 + x_2^2 + \cdots + x_n^2\right)\right]}{2l^2\left(e^2 + \dfrac{I}{A} + \dfrac{EI}{E_t A_t}\right)} \tag{A1.102}$$

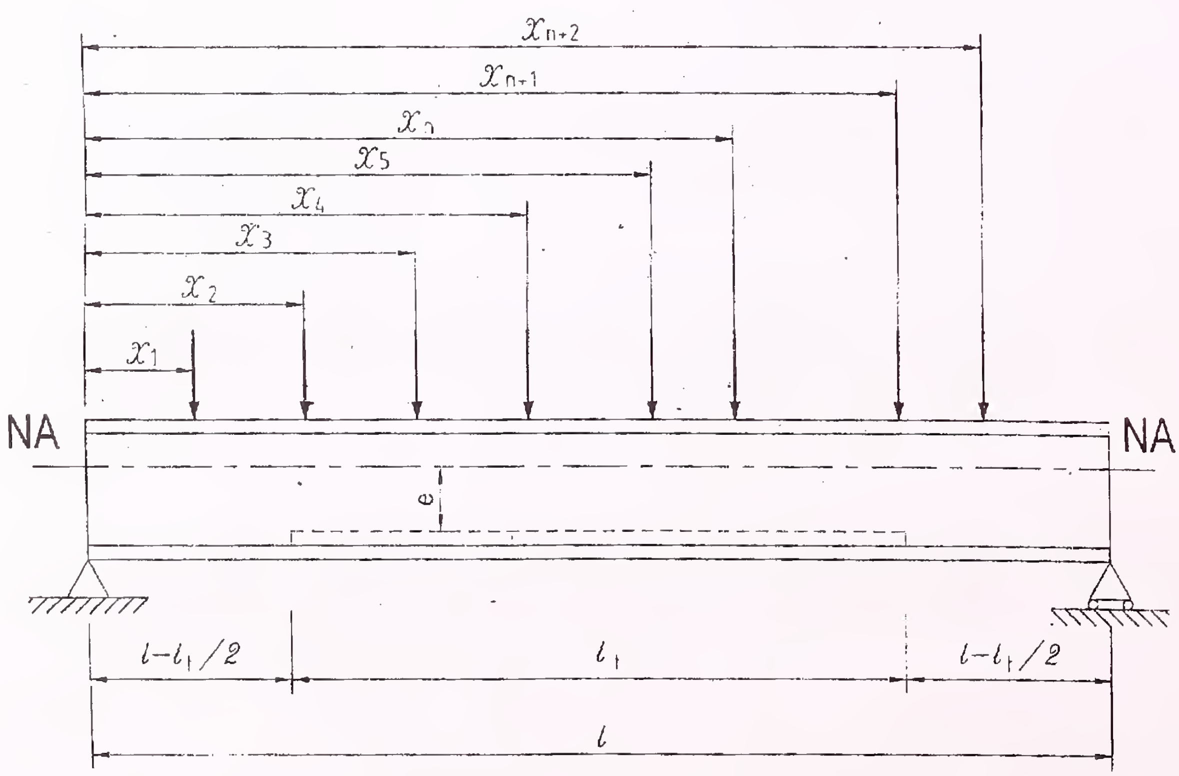


Fig. A1.8.04 Different concentrated moving loads.

7. For the case of different concentrated moving loads (Fig. A1.8.04), we consider the summary of expressions (A1.95), (A1.91) and (A1.97), or

$$\Delta X_4 = \sum \frac{e\left(P_1 x_1 + P_2 x_2 + \cdots\right)}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)}$$

$$\Delta X_3 = \sum \frac{e\left[l\left(P_3 x_3 + P_4 x_4 + \cdots + P_n x_n\right) - \left(P_3 x_3^2 + P_4 x_4^2 + \cdots + P_n x_n^2\right) - (n-2)\left(\frac{l - l_t}{2}\right)^2\right]}{2 l_t \left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)}$$

$$\Delta X_6 = \sum \frac{e\left[P_{n+1}\left(l - x_{n-1}\right) + P_{n+2}\left(1 - x_{n+2}\right)\right]}{2\left(e^2 + \frac{I}{A} + \frac{EI}{E_t A_t}\right)}$$

Annexure - 2

ANALYSIS AND DESIGN OF A PRESTRESSED TRUSS

(Notations used are different from those in text)

Introduction

Static Truss analysis may be performed in two cases, namely, individually prestressed truss members and those trusses with tendons that prestress several members individually.

A2.1 Prestressing of Individual Members

In the first stage, the truss is analyzed under a given loading without considering prestressing. In the second stage, those members acting in tension are designed considering their prestressing.

The cross section of the prestressed members is designed by applying the intensity of the calculated force as follows. Assuming that the member is not loaded by external forces (own weight and live load), the general expressions between the prestress and cross-sectional areas of the member and tendon ay be written as follows:

$$X = -Af = A_c f_c \qquad \text{(A2.01)}$$

where X = the prestressing force

A = the cross-sectional area for a prestressed member

A_c = a cross sectional area of the tendon

f = the compressive stress in the cross-sectional area of a prestressed member

f_c = the tensile normal stress in the tendon

From equation (A2.01), the stresses in the member and tendon are:

$$f = -\frac{X}{A} \qquad f_c = \frac{X}{A_c} \qquad \text{(A2.02)}$$

The stresses given by equation (A2.03) are only those stresses without an external load. By applying an external axial tensile load, F, the stresses in the member and tendon become

$$f = -\frac{X}{A} + \frac{F_m}{A} \qquad f_c = \frac{X}{A_c} + \frac{F_c}{A_c} \qquad \text{(A2.03)}$$

where F_m = the axial force in a member due to external load

F_c = the axial force in a tendon due to external load

The following condition should be satisfied

$$F = F_m + F_c \qquad \text{(A2.04)}$$

The elongations of the prestressed member and tendon should be equal. According to Hook's law, we have

$$\frac{F_m l}{EA} = \frac{F_c l}{E_c A_c} \qquad \text{(A2.05)}$$

where l = the length of a connected prestressed member and tendon

E, E_c = the moduli of elasticity of a member and tendon, respectively

By designating with $\alpha = EA/E_c A_c$, we obtain from equations (A2.04) and (A2.05) those forces F and F_c which are taken by the member and tendon.

$$\frac{EA}{E_c A_c} = \frac{F_m}{F_c} = \alpha \qquad \text{(A2.06)}$$

and $\quad F_m = \alpha F_c = \alpha(F - F_m)$

Therefore

$$F_m = F\frac{\alpha}{1+\alpha} \qquad \text{(A2.07)}$$

and

$$F_c = F - F\frac{\alpha}{1+\alpha} = F\frac{1}{1+\alpha} \qquad \text{(A2.08)}$$

Due to external tensile force F, a tensile stress originates in the member which reduces the magnitude of force F_m. In the tendon, the stresses do not change signs, but rather increase due to the tensile force F. Since the member and tendon elongation should be equal, we obtain

$$\frac{(F - \Delta X)l}{EA} = \frac{\Delta Xl}{E_c A_c} \qquad \text{(A2.09)}$$

where ΔX = an increase in the tendon force X due to the external load.

From equation (A2.09), we have

$$F - \Delta X = \Delta X\frac{EA}{E_c A_c} = \Delta X\alpha$$

from which we obtain

$$\Delta X = F\frac{1}{1+\alpha} \qquad \text{(A2.10)}$$

By using summary of those member and forces which could be expressed by the force F and prestressing force X, we obtain

$$F_m = F - X - \Delta X \qquad F_c = X + \Delta X$$

then the total corresponding stresses should satisfy the condition

$$f = \frac{F - X - \Delta X}{A} \le f_{all} \qquad \text{(A2.11)}$$

$$f = \frac{X + \Delta X}{A_c} \le f_{c,all} \qquad \text{(A2.12)}$$

where $f_{all} = f_y/v$ = the allowable stress of the prestressed member

$f_{c,all} = f_t/v_t$ = the allowable tendon stress

f_y = the yield limit of steel for the prestressed members

f_t = the limiting prestressed tendon tensions

v = the safety coefficient of prestressed member

v_t = the safety coefficient of the tendon

By substituting the expression for X from equation (A2.10) into equations (A2.11) and (A2.12) and considering the relaxation of the tendon's steel, we obtain the following expression for the determination of the prestressed member and tendon stresses.

$$f = -\frac{\psi X}{A} + \frac{F\alpha}{(1+\alpha)A} \le f_{all} \qquad \text{(A2.13)}$$

$$f = \frac{\psi X}{A_c} + \frac{F}{(1+\alpha)A_c} \le f_{c,all} \qquad \text{(A2.14)}$$

The design of the prestressed member, apart from the compressive stresses, may eventually be governed by buckling, which should be checked as follows:

$$\frac{\omega X}{A} < f_{all} \qquad \text{(A2.15)}$$

where ω = the buckling coefficient depending upon the slenderness of the member and type of steel

ψ = the coefficient considering the loss of tendon stress due to relaxation and creep in the tendon

A check of the stress expressed by equation (A2.15) can be performed when the structure is prestressed under such conditions that the member is loaded by external loading.

To obtain the minimum amount of material, the cross-sections of the prestressed members and tendons should be chosen to satisfy conditions $f = f_{all}$ and $f_c = f_{all}$.

By designating $\alpha_0 = \frac{E_c}{E}$ (A2.16)

And substituting the value into equation (A2.06), where

$$\alpha = \frac{EA}{E_c A_c} \qquad \text{(A2.17)}$$

From equations (A2.04), (A2.13), (A2.15), and (A2.17), expressions for the calculation of the required cross sections of the prestressed members, tendons, and prestressing forces may be obtained after certain transformations, as follows:

$$A = \frac{F}{f_{all}} \frac{1 - \alpha_0 \frac{f_{all}}{f_{c,all}}\left(1 + \frac{\psi}{\omega}\right)}{\left(1 - \alpha_0 \frac{f_{all}}{f_{c,all}}\right)\left(1 + \frac{\psi}{\omega}\right)} \qquad \text{(A2.18)}$$

$$A_c = \frac{F - f_{all} A}{f_{c,all}} \qquad \text{(A2.19)}$$

$$X = \frac{1}{\psi}\left(\frac{FA}{\alpha_0 A_c + A} - f_{all} A\right) \qquad \text{(A2.20)}$$

A2.2.1 Prestressing of Individual Members by optimization method

I. Introduction

During the design of separate prestressed steel members of the truss, the following coefficients are used:

a. Coefficient of overloading under prestressing force is $n_1 > 1$ considering the possibility of increase the actual prestressing over design prestressing, and the coefficient $n_2 < 1$,

considering the reduction of actual prestressing force, the loss due to relaxation and yielding of the anchorage.

b. Coefficient of overloading, $n_1 = 1.1$, is used in following two cases:
   - during checking of the member under prestressing, without external loading;
   - during checking of the member when the tresses under external loading coincide by sign with the prestressing.
c. The coefficient $n_2 = 0.9$ is used during checking of the member under external loading, when the stresses are greater and have the reverse sign of the prestressing.
d. In the case of safe and direct control of the prestressing, $n_1 = n_2 = 1$.
e. During design of the member having a tendon of steel cable, and safe and direct control is assured, the values of these coefficients are $n_1 = 1.05$ and $n_2 = 0.95$.

II. Basic Design Assumptions

The rigid member and tendon working together are a statically indeterminate system. For their analysis it is necessary to assume the distribution of the material between rigid member and tendon as follows:

$$K = \frac{A_t}{\sum A} \qquad (1-K) = \frac{A_m}{\sum A} \qquad \sum A = A_t + A_m \tag{A2.21}$$

where A_t and A_m are the cross sectional areas of the tendon and member, respectively.

The required total cross sectional area is

$$\sum A = \frac{F_{tot}}{f_m\left[(1-K) + K\frac{f_t}{f_m}\right]} \tag{A2.22}$$

where F_{tot} = total force acting in the member

f_t, f_m = allowable stresses of the tendon and member, respectively

$$A_t = K\sum A \qquad A_m = (1-K)\sum A \tag{A2.23}$$

The prestressing force is

$$Z = \varphi f_m A_m \tag{A2.24}$$

where φ is the coefficient of the longitudinal bending of the member, used in the range 0.9-0.95.

The force in the tendon acting under total design force under external loads is

$$X_t = \frac{F_{tot} A_t \dfrac{E_t}{E}}{A_t \dfrac{E_t}{E} + A_m} \tag{A2.25}$$

where E_t and E are the moduli of elasticity of the tendon and member, respectively.

The tendon stress is

$$f_t = \frac{Z n_1 + X_t}{A_t} < f_{all} \tag{A2.26}$$

The force in the cross section of the member under loading is

$$F_m = F_{tot} - (Zn_2 + X_t) \qquad \text{(A2.27)}$$

where n_1 and n_2 are the coefficients of overloading and underloading of prestressing of the tendon. Checking of stress in the member under loading,

$$f_m = \frac{F_m}{A_m} \qquad \text{(A2.28)}$$

With the above design method, optimal use of metal or cost of the prestressed member requires repetition of design procedure. However, it is possible to eliminate repetitive design by the following method. Considering the total design carrying capacity of the member and tendon, we may write

$$Zn_1 + \Delta F_{tot} = f_t A_t \qquad \text{(A2.29)}$$

$$-Zn_2 + (F_{tot} - \Delta F_{tot}) = f_m A_m \qquad \text{(A2.30)}$$

Equation (A2.29) relates to the tendon and equation (A2.30) to the member. By solving the system of equations (A2.29) and (A2.30), we obtain the following formulas for the design of a prestressed truss member

$$A_t = \frac{n_1 \varphi \beta \alpha^2 f_m}{\alpha(\beta f_t + n_1 \varphi f_m) - F_{tot}} \qquad \text{(A2.31)}$$

$$A_m = \alpha - \frac{A_t}{\beta} \qquad \text{(A2.32)}$$

$$\alpha = \frac{F_{tot}}{f_m(1 + n_2 \varphi)} \qquad \text{(A2.33)}$$

$$\Delta F_{tot} = \frac{F_{tot} A_t}{A_t + \beta A_m} \qquad \text{(A2.34)}$$

$$Z = \varphi f_m A_m \qquad \text{(A2.35)}$$

where ΔF_{tot} = part of the total force under external loading, acting in the tendon

α = cross-section of the member, reduced to the rigid material

β = E/E_t = ratio of moduli of elasticity of the member and tendon

Knowing the total design force acting in the member and choosing the material, we may obtain, using formulas (A2.31) and (A2.35), cross-sectional areas of the member and tendon and also prestressing force.

From expression (A2.32) it follows that it is possible to realize the prestressing only when $A_m > 0$, or

$$\alpha - \frac{A_t}{\beta} > 0 \qquad \text{(A2.36)}$$

By introduction the denominations for ratios of design stresses of the tendon and member $K_1 = f_t / f_m$, and after certain transformations of formula (A2.36) we obtain the condition of possible prestressing of the member

$$K_1 > \frac{1 + \varphi n_2}{\beta} \qquad \text{(A2.37)}$$

and after substitution value of α, we have

$$K_1 > \frac{1+\varphi n_2}{\beta} \qquad \text{(A2.38)}$$

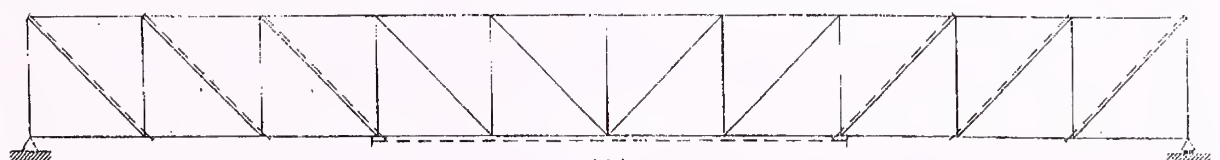
At $\beta = 1$, $\varphi = 1$, and $n_2 = 0.9$, the minimum value of ratio $K \geq 1.9$. At smaller φ the ratio of K_1 also diminishes.

We intend to evaluate how the effect of prestressing of the members influences the change of the design stress ratio K_1 and the coefficient of longitudinal bending φ, or more exactly, flexibility of member. For this purpose we introduce into formula (A2.31) the value K_1, and certain transformations we obtain the following expression for cross-sectional area of the tendon $A_t = \dfrac{n_1 \varphi \beta F_{tot}}{f_m (1 + n_2 \varphi)[\beta K_1 + (n_1 - n_2)\varphi - 1]}$ (A2.39)

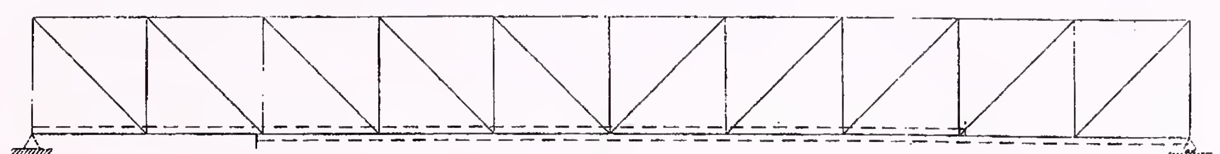
A2.2 A TRUSS HAVING A BOTTOM CHORD STRENGTHENED BY TENDONS

A2.2.1 General

Prestressed trusses are considered as statically indeterminate systems where an increase of the tendon force is taken as an additional unknown. Applying the usual prestressing method with a straight tendon, the magnitude of the tendon force depends on the assumed cross section of a truss member, its external load, and the tendon cross section (Fig. A2.01)



(A)



(B)

Fig. A2.01 Prestressed trusses.
(A) Prestressed single diagonals and bottom chord.
(B) Prestressing of a bottom chord with two tendons.

The first condition for the prestressed member i is

$$F_{i,x} \leq \frac{F_{all} A_i}{\omega} \qquad \text{(A2.40)}$$

where $F_{i,x}$ = a force in member i under prestressing force X

F_{all} = the allowable stress in a member

ω = the buckling coefficient

A_i = the cross-sectional area of a prestressed member, i

The second condition for a prestressed member is

$$X + \Delta X \leq f_t A_t \qquad \text{(A2.41)}$$

where f_t = the allowable tendon stress

A_i	=	the cross-sectional tendon area
X	=	the magnitude of an assumed prestress force
ΔX	=	an increase in the tendon force under external load

ΔX can be calculated by using the Maxwell-Mohr principle.

For a simple span truss having a single tendon parallel to its bottom chord, the magnitude of ΔX is

$$\Delta X = \frac{\sum_i \frac{F_{i,x=1} F_{i,q} \times l_i}{EA_i}}{\sum_i \frac{(F_{i,x=1})^2 l_i}{EA_i} + \frac{l_t}{E_t A_t}} \qquad \text{(A2.42)}$$

where $F_{i,x=1}$	=	a force in member i due to a prestress force $X = 1$
$F_{i,q}$	=	a force in member i due to an external load q
l_i	=	the length of the member
EA_i	=	the stiffness of the member
l_t	=	the length of tendon
E_t	=	the modulus of elasticity of the tendon
A_t	=	the cross-sectional area of the tendon

The values of $F_{i,x=1}$ and $F_{i,q}$ are determined by conventional methods.

A2.2 DEFLECTION OF THE TRUSS

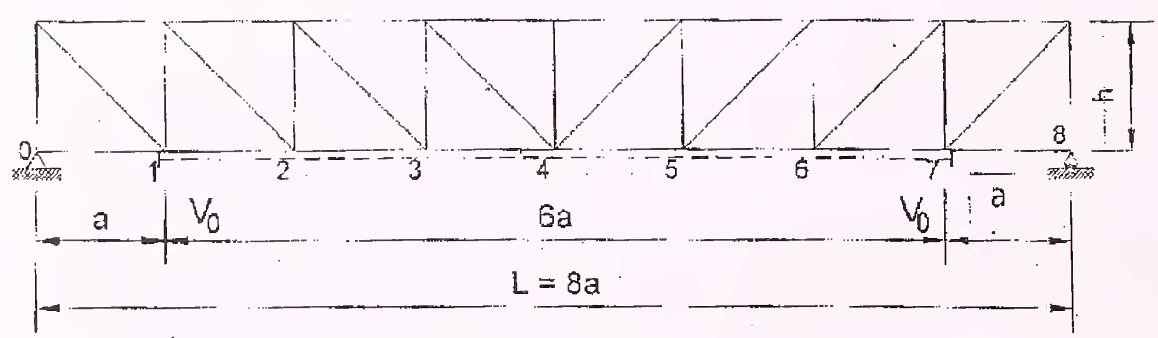


Fig. A2.02 A truss having a tendon shorter than that of its span

The determination of a deflection due to tendon force X_0 will be shown in an example of one truss which is prestress by a tendon located at its bottom chord, and having a length less than that of its span (Fig. A2.02). The deflection will be determined by using Maxwell-Mohr rule, which is

$$f = \sum \frac{S_{i,l} S_{i,x} l_i}{EA_i} \qquad \text{(A2.43)}$$

where f	=	the deflection of a truss at the joint under consideration
$S_{i,l}$	=	a force in i, due to unit force $P = 1$ at this joint, for which the deflection is required
$S_{i,x}$	=	a force in member i, due to X

l_i	=	the length of member i
A_i	=	a cross-section of member i
E	=	the modulus of elasticity of the member

An example is given in Table A2.01, where the force $S_{i,i}$ and $S_{i,x}$ which are in the truss members due to loading are shown at joint 4 under force $P_4 = 1$ in Fig. A2.02. The bottom chord along the whole span has a constant cross section.

The maximum deflection at the middle joint of the bottom chord is

$$f = -4\frac{a^2 X}{hEA} - 2x\frac{2a^2 X}{hEA} = -8\frac{a^2 X}{hEA} \qquad \text{(A2.44)}$$

The total deflection of the truss is the sum of the deflection of the truss due to the influence of the tendon force and the deflection due to external loading, which act as concentrated forces at the joints.

Table A2.01 Forces in the Truss

Member	Forces	1-2	2-3	3-4	4-5	5-6	6-7
Force $S_{i,x}$ in member	Due to force in tendon	$-X$	$-X$	$-X$	$-X$	$-X$	$-X$
Force $S_{i,i}$ in member	Due to unit force	$\frac{a}{h}$	$\frac{a}{h}$	$\frac{2a}{h}$	$\frac{2a}{h}$	$\frac{a}{h}$	$\frac{a}{h}$

To calculate the optimum carrying capacity, the tendon is located underneath of the bottom chord. In this case for many members of the truss, the required cross-sectional areas may be reduced. Irrespective of slight stiffness of truss members and irrespective of large loading, the total deflection of one prestressed truss is generally smaller than the deflection of the equal but not prestressed truss.

Annexure - 3

Important Formulae for Prestressed Steel Beam

(Refer Annexere-1)

1. Parameter characterizing asymmetry of an I-Girder

$$a = \frac{h_2}{h_1} = \frac{S_1}{S_2}$$

choose "a" in the range of 1.5 to 2.0

2. Parameter characterizing web flexibility

$$K = \frac{h}{t_w}$$

choose "K" in the range of 100 to 200

3. Parameter characterizing material distribution

$$m = \frac{A_w}{A}$$

choose "m" in the range of 0.5 to 0.6

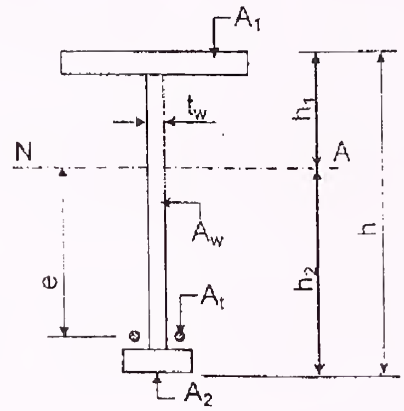


Fig A3.01

4. $$m = \frac{6a[a-(1+\beta)]}{(a+1)[a(1-\beta)-(1+\beta)]}$$

5. $$a = \frac{m\beta - 3(1+\beta) - \sqrt{m^2 - 6m(1+\beta)^2 + 9(1+\beta)^2}}{m(1-\beta)-6}$$

6. $$C = (1+\beta)\sqrt{\frac{6a^3(1-a)^2[a-(1+\beta)]}{(a+1)^3[a(1-\beta)-(1+\beta)]^3}}$$

when $\beta = 1$, $a = 1.71$, and $m = 0.55$, $C = 0.33$

7. $$A = \sqrt[3]{\frac{M^2}{C^2 f^2 K}}$$

8. $$A_1 = A\left(\frac{a}{a+1} - \frac{m}{2}\right)$$

9. $$A_2 = A\left(\frac{1}{a+1} - \frac{m}{2}\right)$$

10. $$A_w = m\,A = h t_w = \frac{h^2}{K}$$

11. $$A = A_1 + A_2 + A_w$$

12. $$A_t = \frac{A}{f_t} . f_{all}\left(\frac{a-1}{a+1}\right)$$

13. $$h \approx \sqrt{AKm}$$

14. $$h_1 = \frac{\sqrt{AKm}}{1+a}$$

15. $$h_2 = \frac{a\sqrt{AKm}}{1+a}$$

16. $$I_x = A^2 Km \frac{6a-(a+1)^2 m}{6(a+1)^2} \quad \text{from} \quad I_x = \frac{t_w h^3}{12} + t_w h\left(h_2 - \frac{h}{2}\right)^2 + A_1 h_1^2 + A_2 h_2^2$$

Symbols:

A_1	= Area of top flange
A_2	= Area of bottom flange
A_w	= Area of web
A	= Total cross sectional area of girder
A_t	= Cross sectional area of tendon
E_m	= Elastic Modulus of member
E_t	= Elastic Modulus of tendon
S_1	= Section modulus for compressed edge
S_2	= Section modulus for tensioned edge
X	= Prestressing Force
ΔX	= Increment in prestressing force
M	= Bending Mom due to External Loading
I_x	= Moment of Inertia of girder about N.A.
f_{all}	= Allowable stress in structural steel
f_t	= Allowable stress in tendon chords
e	= eccentricity of tendon with respect to neutral axis of girder cross section
β	$= \frac{X + \Delta X}{X}$
ψ	= coeff of buckling
a	= Parameter characterizing asymmetry of an I-Girder
m	= Parameter characterizing material distribution

17. Section Modulus $S_1 = \sqrt{A^3Km}\,\frac{6a-(a+1)^2m}{6(a+1)}$

18. Section Modulus $S_2 = \sqrt{A^3Km}\,\frac{6a-(a+1)^2m}{6a(a+1)}$

19. Area of cross section of Girder A $= \sqrt[3]{\frac{M^2}{f^2K}}\left[\frac{(a+1)}{a^3\sqrt{3(1-a)^2(2-a)}}\right]^{-1/3}$

20. Prestressing Force hall be smaller of the two given below

$$X = \frac{FA[6a-(a+1)^2m]}{(a+1)[6a-(a+1)m]} \quad \text{and} \quad X = \frac{\Psi FAS_2}{S_2 + e.A}$$

Where F = allowable stress in structural steel

ψF = Allowable buckling stress in bottom flange

21. Self stressing Force $\Delta X = \frac{2Me}{3\left(e^2 + \frac{I}{A} + \frac{EI}{E_tA_t}\right)}\left(2 - \frac{L_t}{L}\right)$ for uniformly distributed loading

Where L = Length of beam ; L_t = Length of prestressing tendon

22. Self stressing Force $\Delta X = \sum \frac{Pe(L.x_1 - x_1^2 - a^2)}{2(l-2a)\left(e^2 + \frac{I}{A} + \frac{EI}{E_tA_t}\right)}$

Under series of equal concentrated loads P acting at x_1, x_2, x_3 .... from support

Where L = Length of beam ; L_t = Length of prestressing tendon ; a = $(L - L_t)/2$

23. Upward Deflection due to Prestressing Force $(X+\Delta X)$ is given by,

$$\delta_{Prestress} = -\frac{(X+\Delta X)eL^2}{8EI}\left[1 - 4\left(\frac{a}{L}\right)^2\right]$$

Where L = Length of the beam ; a = $(L - L_t)/2$

24. Downward deflection due to equivalent uniformly distributed load,

$$\delta = +\frac{ML^2}{10EI}$$

25. Natural Frequency of vibration n of girder is given by

$$n = 1.57\sqrt{\frac{E.I.g}{wl^4}}$$

26. Period of Natural Vibration of Girder T $= 0.637\sqrt{\frac{wl^4}{E.I.g}}$

Important Formulae for Prestressing of Individual Truss Member by Optimization method

(Refer Annexure 2)

1. $r = \dfrac{A_t}{\sum A}$; $(1-r) = \dfrac{A_m}{\sum A}$; $\sum A = A_t + A_m$; $\alpha = \dfrac{E_m A_m}{E_t A_t}$; $\beta = \dfrac{E_m}{E_t}$

   where A_t = cross sectional area of tendon
   A_m = cross sectional area of member
   E_t = Modulus of elasticity of tendon
   E_m = Modulus of Elasticity of member

2. The required total cross sectional area is

$$\sum A = \frac{F_{total}}{f_m\left[(1-r) + r.\dfrac{f_t}{f_m}\right]}$$

where F_{total} = Design Force acting on the member
f_t = allowable stress in tendon
f_m = allowable stress in steel member

3. The prestressing force is given by
   $Z = \psi f_m A_m$; ψ = co-efficient for buckling used in the range 0.9 to 0.95

4. The force in the tendon acting under total design force under external load is,

$$X_t = \frac{F_{total}.A_t.\dfrac{E_t}{E_m}}{A_t.\dfrac{E_t}{E_m} + A_m}$$

5. For optimization the following two conditions must be satisfied

   (i) $Z.n_1 + \Delta F_{total} \le f_t.A_t$
   (ii) $-Z.n_2 + (F_{total} - \Delta F_{total}) \le f_m.A_m$

   where n_1 is the overloading factor 1.1
   n_2 is the under loading factor 0.9

6. $$A_t = \frac{n_1 \psi \beta \alpha^2 f_m}{\alpha(\beta f_t + n_1 \psi f_m) - F_{total}} = \frac{n_1 \psi \beta F_{total}}{f_m (1 + n_2 \psi)[\beta k + (n_1 - n_2)\psi - 1]}$$

   where $k = \dfrac{f_t}{f_m}$; $A_m = \alpha - \dfrac{A_t}{\beta}$; $\alpha = \dfrac{F_{total}}{f_m(1 + n_2\psi)}$

   also as $A_m > 0$, $\therefore$ $\alpha - \dfrac{A_t}{\beta} > 0$

7. $$\Delta F_{total} = \frac{F_{total}\, A_t}{A_t + \beta A_m}$$

   where, ΔF_{total} = Part of total force under external loading, acting in tendon

8. Condition of possible prestressing is satisfied when

   $k \ge \dfrac{F_{total}}{\alpha \beta f_m}$ also $k \ge \dfrac{1 + \psi.n_2}{\beta}$

Prestressed Steel Road Bridges

Numerical Example 1: Non composite prestressed steel girder

Design a prestressed inner steel plate girder with the following data:

Effective Span : 24.00 m Girder Spacing : 3 m c/c

M_{DL} =	1650.00 kN-m	V_{DL} =	280.00 kN
M_{SIDL} =	550.00 kN-m	V_{SIDL} =	100.00 kN
$M_{LL+Impact}$ =	1450.00 kN-m	$V_{LL+Impact}$ =	380.00 kN

Material properties :

Steel Grade :	Fe540B High Tensile
(f) $f_{allowable}$:	230.0 N/mm^2
E_s :	200000 N/mm^2
Poisson's Ratio μ :	0.30
High tensile wires for prestressing :	
f_t :	950.0 N/mm^2
E_t :	160000 N/mm^2

The girder shall be designed as a non-composite prestressed beam, using working stress method.

Limiting deflection : Δ/span = 0.00167

Solution :

1. Maximum design bending moment M_{max} = 3650.0 kN-m
Maximum design shear force V_{max} = 760.0 kN

CHOICE OF CROSS SECTION :

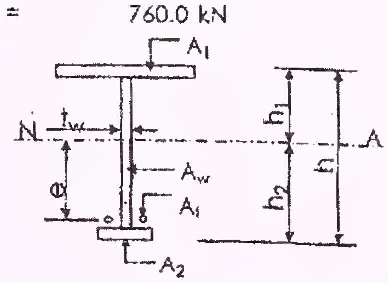


Fig A3.02

Assuming	$a = h_2/h_1$ =	1.87
	$K = h/t_w$ =	120
	$m = A_w/A$ = /	0.537
	$\beta = (X+\Delta X)/X$ =	1.0
	C=	0.33

Area of section $A = (M^2/(f^2K))^{1/3} * (a+1)/(a(3*(1-a)^2(2-a))^{1/3})$
= (3650000000^2/(230^2*120))^(1/3)
(1.87+1)/(1.87(3*(1-1.87)^2*(2-1.87))^(1/3)) =
29511.34 mm^2

$h_2 = a(AKm)^{1/2}/(a+1)$ = 1.87*(29511.34*120*0.537)^0.5/(1.87+1) =
898.53 cm say 899.00 mm

h_1 =	h_2/a =	899.0/1.87 =	480.75 mm
h =	$h_1 + h_2$ =	480.75+899.00 =	1379.75 mm
t_w =	h/K =	1379.75/120 =	11.50 mm

use 12 mm thick web

$A_1 = A(a/(a+1)-m/2)$ = 29511.34*(1.87/(1.87+1)-0.537/2) = 11304.85 mm^2

$A_2 = A(1/(a+1)-m/2)$ = 29511.34*(1/(1.87+1)-0.537/2) = 2358.90 mm^2

A_w = 29511.34-11304.85-2358.90 15847.59 cm^2

Assuming centre of prestressing tendon is 100 mm above bottom flange.

So, eccentricity e = 898.53-100 = 798.53 cm

Numerical Examples

Metallic area of tendon $A_t = A.f_{allowable}/f_t(a-1)/(a+1)$
$= 32180*230/950*(1.87-1)/(1.87+1) =$
$= 2361.72\ mm^2$

Using 7 mm wires, tendon cross section = a_w = 3.1416/4*7^2
= 38.48 mm^2

Hence nos. of wires required = 2362/38 = 61 Nos. of 7mm wires
For practical purpose use 2 Nos. BBRV cables each containing 36 wires.
Metallic area of cable A_t = 2770.88 mm^2
Maximum prestressing force allowed = 2770.88*950/1000 = 2742.02 kN
or use 2-12T13 Prestressing strands

2. Geometric Properties of chosen cross section of steel girder :
Section chosen : (all dimentions are in mm)

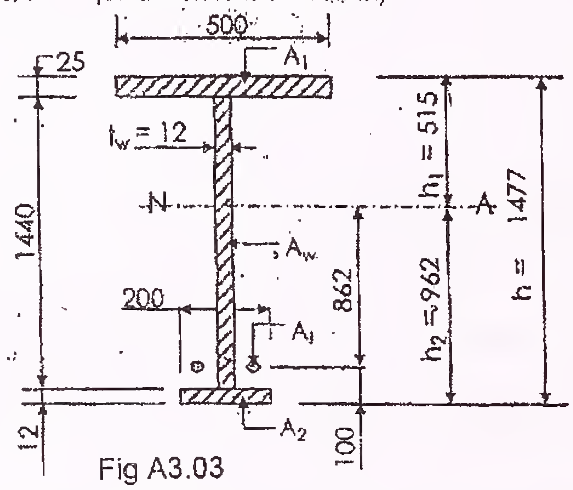


Fig A3.03

<u>Calculation of sectional properties</u> (excluding tendon)

Area : 200*12+1440*12+25*500 = 32180 mm^2
Depth of C.G. from top fibre:
h_1 = (500*25*12.5+1440*12*745+12*200*1471)/32180 = 514.61 mm
So, h_2 = 962.39 mm
I_{xx} = 500*25^3/12+500*25*502.113113735239^2+12*1440^3
/12+12*1440*230.386886264761^2+200*12^3/12+200*
12*956.386886264761^2 = 9.251E+09 mm^4

Section modulus (top) S_1 = 9250545550/514.6 = 17975728 mm^3
Section modulus (bottom) S_2 = 9250545550/962.4 = 9612086 mm^3
Eccentricity of the cable, e = (962.39-100) = 862.39 mm

K= h/t_w = 120.0 m =A_w/A= 0.537 a =S_1/S_2 = 1.87

Numerical Examples

3. Area of tendon A_t= 32180*230/950*(1.87-1)/(1.87+1) = 2361.94 mm^2 < 2770.88cm^2, OK

Prestressng force = 950*2361.94e-3 = 2243.84 kN

4. Determnation of Prestressng Force wth respect to stabilty of bottom flange :

For the bottom flange,
I_y = 12*200^3/12 = 8000000 mm^4
r_y = (8000000.0/2400)^0.5 = 57.7 mm
S_b = (8000000.0/(200/2)) = 80000 mm^3
A_b = 12*200 2400 mm^2

Assuming the spacng of points fastening of the tendon to the bottom flange of the beam equals = 1000 mm c/c

λ_y = 1000/57.74 = 17.32 From Fig A1.7.01 (Annexure - 1)
Value of reduction factor ψ (corresponding to λ_y) = 0.96

The value of permissible prestressing force (limited by buckling stress in bottom flange) is given by:

$$X = \frac{\psi\ f_{all}\ S_2\ A}{S_2 + e\ A} = \frac{0.96 * 230 * 9612086 * 32180e\text{-}3}{9612086 + 862.39 * 32180} = 1827.9\ kN$$

5. Length of tendon after formula

α = 1-$m^{0.5}$/C*(6a-m(a+1)^2)/(6a(a+1)) = 0.531

Length of the tendon L_t = L*(α)^0.5 = 17.49 m say 18.00 m
So, anchorage length L_a = (L-L_t)/2 = 3 m

Moment of resistance M_R for 3.00 m length from either support of the beam is given by, M_R = f_{all}*S_2 = 2210.8 kN-m

6. General Equation of Prestressing Force is given by formula

$$X = \frac{f_{all}.A[6c-(a+1)^2 m]}{(a+1)(6a-(a+1)m)}$$
$$= \frac{230*32180*((6*1.87-1.87+1).\verb|^|2*0.537)*1e\text{-}3}{(1.87+1)*(6*1.87-(1.87+1)*0.537)} = 1810.91\ kN$$

Adopted prestressing force X= 1810.91 t

The value of increament in prestressing force under equivalent distributed load is given by formula

ΔX = 2M_P.e/(3*(e^2+I/A+EI/($E_t A_t$)))*(2-L_t/L)
= 2*3650*1e+06*862.39*1e-3*(2-18/24)/(3*(862.4^2+ (9250545550/32180)+(200000*9250545550/(160000*27 70.88) = 504.03 kN
say 505.00 kN

Numerica Examples

7. Check for tendon strength

$\sigma_t = (X+\Delta X)/A_t =$ (1810.9+505)*1e+03/2770.88 = 835.80 N/mm^2

< 950.00 N/mm^2

Hence OK

8. Checking of bending stresses in the steel girder :

(a) In the course of prestressing :

tension (f_{all})

f_{top} = -1810.91*1e+03/32180+1810.91.1e+03*862.39/17975728 =

compression (ψ f_{all}) 30.60 N/mm^2

f_{bot} = -1810.91*1e+03/32180-1810.911e+03*862.39/9612086 =

-218.75 N/mm^2

< allowable stress in steel, OK

b) Due to loaded span :

f_{top} = $= -(X+\Delta X)/A - (M-(X+\Delta X)e)/S_1$ = -163.91 N/mm^2

f_{bot} = $= -(X+\Delta X)/A + (M-(X+\Delta X)e)/S_2$ = 99.98 N/mm^2

< allowable stress in steel, OK

9. Deflection Calculation

(a) $\Delta_{DL+SIDL} \approx$ $ML^2/10EI$ = (1650+550)*1e+06*24000^2/(10*200000*9250545550) =

68.493 mm downward

(b) $\Delta_{LL+Impact} \approx$ = 1450*1e+06*68.49/((1650+550)*1e+06) =

45.143 mm downward

(c) $\Delta_{prestress} \approx$ $= -(X+\Delta X)eL^2(1-4(L_a/L)^2)/8EI$ =

$$= \frac{-(1810.91+505)*1e+03*862.39*24000^2(1-4*(3/24)^2)}{(8*200000*9250545550)}$$

= -72.87 mm upward

Resulting deflection $\Delta = \Sigma\Delta_i$ = 68.49+45.14-72.87 = 40.770 mm downward

Provide longitudinal camber in girder 1 in 400

=-24/2*1000*1/400 = -30 mm to minimize deflection

Net deflection = 10.770 mm downward

So, Δ/Span = 0.00045 < allowable deflection, OK

10. Tangential Shear stress at support :

Maximum Shear at support = 760.00 kN

Static Moment Q = 500*25*502.1+12*489.61*244.81 = 7714739.9 mm^3

Shear Stress τ = $VQ/I_{xx}t_w$ = 760*1e+03*7714739.9/(9250545550*12)=

52.8 N/mm^2

< 0.4*(allowable tensile stress in steel), OK

11. Buckling Stresses :

$f_{cr} = \lambda_v \pi^2 E/(12(1-\mu^2)(h/t_w)^2 =$ 17.32*3.1416^2*200000/(12*(1-0.3^2)*(1440/12)^2)

= 217.4 N/mm^2

$\tau_{cr} = (5.34+4/(a'/b')^2)*\pi^2E/(12(1-\mu^2)*(h/t_w)^2) =$

= (5.34+4/(1000/1440)^2)*(3.1416^2*200000)/(12*(1-0.3^2)*(1440/12)^2) =

171.2 N/mm^2

Interaction Ratio for buckling stress = $((f_{max}/f_{cr})^2+(\tau_{max}/\tau_{cr})^2)^{0.5}$ =

= ((163.9/217.4)^2+(52.8/171.2)^2)^0.5

= 0.815

< 1, OK

Numerical Examples

Prestressed Steel Road Bridges

Numerical Example 2: Composite prestressed steel girder

Design a composite prestressed steel girder with the following data :

Effective Span :	24.00 m	Girder Spacing :	3 m c/c
Deck slab thickness :	200 mm	Grade of concrete:	M 35
M_{DL} =	1650.00 kN-m	V_{DL} =	280.00 kN
M_{SIDL} =	550.00 kN-m	V_{SIDL} =	100.00 kN
$M_{LL+Impact}$=	1800.00 kN-m	$V_{LL+Impact}$=	380.00 kN

Shored construction using temporary support is to be used.
(entire loading i.e. DL+SIDL+LL+Impact is taken by the composite prestressed girder)

Material properties :

Steel Grade :	Fe540B High Tensile
$f_{allowable}$:	230.00 N/mm^2
E_s :	200000 N/mm^2
Poisson's Ratio μ :	0.30
High tensile wires for prestressing :	
f_t :	950.00 N/mm^2
E_t :	160000 N/mm^2
Concrete:	
Allowable bending compression stress in concrete f_{bc} :	10 N/mm^2
Modular ratio for transient loading : n_1 =	7.5
Modular ratio for permanent loading : n_2 =	15

The girder shall be designed as a composite prestressed beam, in working stress method.

Limiting deflection : δ/span = 0.00167

Solution :

1. Maximum design bending moment M_{max} = 4000.00 kN-m
   Maximum design shear force V_{max} = 760.00 kN

2. Determination of Steel Cross Section for Composite action:

When all moments are carried by the top and bottom flanges of the steel section along with the concrete slab, the following equations may be written:

$(M_{DL} + M_{SIDL}) / (H_S A_b) + M_{LL+I} / (H_{CP} A_b) = f_{all}$

$(M_{DL} + M_{SIDL}) / (H_S A_t) + M_{LL+I} / (H_{CP}(A_t + A_c)) = f_{all}$

where H_S Height of the steel section

H_{CP} Height of the composite section

As an approximation assume 20% of M is carried only by web.

Then the required cross sectional areas of web & flanges :

M_{web} = 20/100*4000 = 800 kN-m

Z_{web} = 800*1e+06/230 = 3478261 mm^3

assuming 12 thk web, depth of web = 1318.8 mm

say 1320.0 mm

So, that K = d/t_w = 1320/12 = 110

Using A_w / A = 0.55 ;

A = 1320*12/0.55 = 28800 mm^3

Assume as a first approximation, tendon area = 2% of steel area

A_T = 576 mm^3

We assume that the balance 80% BM is resisted by the bottom flange and reinforced concrete slab
Rearranging the above equations and simplifying, we compute N_1, N_2, A_b and A_t as shown below:

Normal Thrust $N_1 = 0.80M_1/H_s =$ 0.80*(1650+550)*1e+03/(1320)= 1333.3 kN
Normal Thrust $N_2 = 0.80M_2/H_{CP} =$ 0.80*1800*1e+03/(1320+100) = 1014.1 kN

(a) Bottom flange :

$A_{b'} = (N_1+N_2)/f_{all} =$ (1333.3+1014.1)*1e+03/230 = 10206.2 mm^2
using tendon to reduce bottom flange steel requirement:
$A_b = A_{b'} - A_T{}^*f_t/f_{all} =$ 10206.2-576*950/230 = 7827.0 mm^2

(b) Top flange :

A_C = Transformed area of concrete section (for permanent loading) =
= 3000*200/15 = 40000 mm^2
using notation A_R as defined below,
$A_R = A_C - (N_1 - N_2)/f_{all}$ 40000-(1333.3-1014.1)*1e+03/230 = 38612.0 mm^2
$A_t = (N_1A_C/f_{all} + 0.25\,A_R{}^2)^{0.5} - 0.5\,A_R =$
= (1333.3*40000*1e+03/230+0.25*38612.0^2)^0.5-0.5*38612.0 =
5282.7 mm^2

3. Properties of Composite cross section, (n = 15)

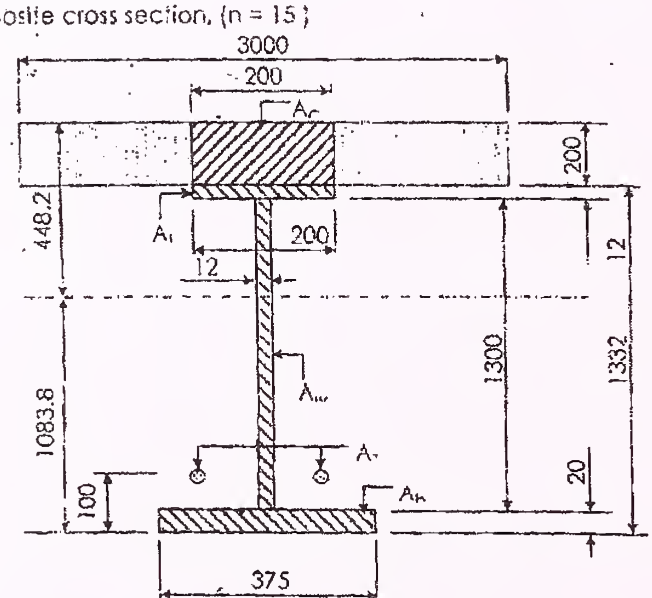


Fig A3.04

c/s Area A = 200*200+12*200+12*1300+20*375 = 65500 mm^2
C.G. distance from top fibre
=(200*200*100+200*12*206+12*1300*862+375*20*1522)/65500 = 448.19 mm

$I_{CP} =$ 1/3*(200*448.2^3-2*94*236.2^3+375*1083.8^3-2*181.5*1063.8^3) =
= 1.86E+10 mm^4

S_1 = 18640170910/448.2 = 41589666 mm^3
S_2 = 18640170910/1083.8 = 17198782 mm^3

A' (steel section only) = 25500 mm^2
A_t = 2400 mm^2
A_w = 15600 mm^2
A_b = 7500 mm^2
A_C (Transformed section) = 40000 mm^2

Eccentricity of tendon e = 1083.8-100 = 983.8 mm

Numerical Examples

For the bottom flange.	I_y =	20*375^3/12 =	87890625 mm^4
	r_y =	(87890625.0/7500)^0.5 =	108.3 mm
	S_b =	(87890625.0/187.5) =	468750 mm^3
	A_b =	375*20	7500 mm^2

Assuming the spacng of points fastening of the tendon to the bottom flange of the beam equals = 1000 mm c/c

λ_y = 1000/108.25 = 9.24 From Fig A1.7.01 (Annexure - 1)

Value of reduction factor ψ (corresponding to λ_y) = 0.98

The maximum value of permissible prestressing force(based on bottom flange buckling) is given by:

$$X = \frac{\psi\, f_{all}\, S_2\, A}{S_2 + e\, A} = \frac{0.98 * 230 * 17198782 * 65500 * 1e\text{-}3}{17198782 + 983.81 * 65500} = 3110 \text{ kN}$$

Taking tendon area at 2% of the steel cross section, apply prestressing force

X = 576*950/1000 = 547.2 kN after concrete hardens,

so that the composite action is available.

STAGE 1:

The steel girder is propped at 2 locations namely at 1/3L and at 2/3L and is strong enough to carry the dead load stresses alone. A stress check can be made to see that the stresses are within permissible limits.

Here M = 1650*(8/24)^2 =	183.33 kN-m
S_{min} of the steel section alone =	3150310 mm^3
So, the stress in the steel section alone =	58.20 N/mm^2

STAGE 2:

Prestressing applied after concrete hardens. This will lift the girder from temporary support and the entire dead load moment will be caried by composite action of steel girder. Since it is a long term effect, modular ratio used should be n = 15

(a) Stress at top

$f_{cb}^{t,2}$ = $-X/A_{CP}-(M_{DL}-X.e)/S_1$

= -547.2*1e+03/65500-(1650*1e+06-547.2*1e+03*983.8)/41589666.2 =

-35.08 N/mm^2 (in steel units) or -2.34 N/mm^2 (in concrete units)

(b) Stress at bottom

$f_{cb}^{b,2}$ = $-X/A_{CP}+(M_{DL}-X.e)/S_2$

= -547.2*1e+03/65500+(1650*1e+06-547.2*1e+03*983.8)/17198781.7 =

56.28 N/mm^2 (in steel units)

STAGE 3: Stresses due to superimposed dead load. (n = 15)

(a) Stress at top $f_{cb}^{t,3}$ = $-M_{SIDL}/S_1$

= -550*1e+06/41589666.2 =

= -13.22 N/mm^2 (in steel units) or

= -0.88 N/mm^2 (in concrete units)

Numerical Examples

(b) Stress at bottom $f_{cp}^{b,3}$ = M_{SIDL}/S_2

= 550*1e+06/17198781.7 =

= 31.98 N/mm^2 (in steel units)

STAGE 4: Stresses due to live load + impact

Since this being transient short term effect, n =7.5

Properties of Composite cross section, (n = 7.5)

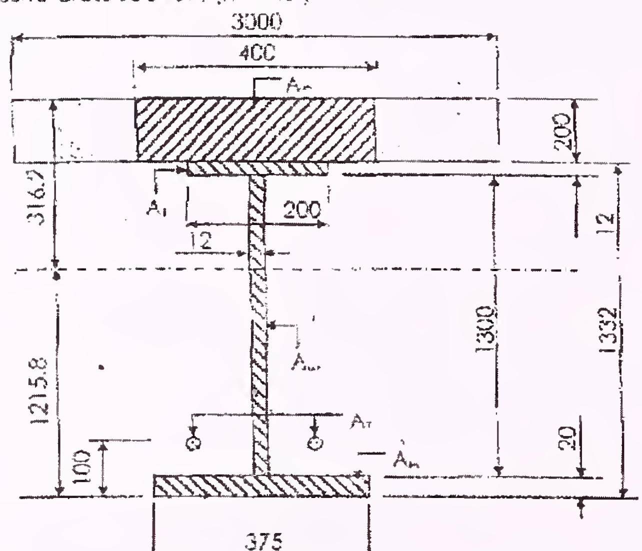


Fig A3.05

c/s Area A = 400*200+12*200+12*1300+20*375 = 105500 mm^2

C.G. distance from top fibre

=(400*200*100+200*12*206+12*1300*862+375*20*1522)/105500 = 316.18 mm

I_{cr} = 1/3*(400*316.2^3-2*194*104.2^3+375*1215.8^3-2*181.5*1195.8^3) =

= 2.18E+10 mm^4

S_1 = 21813503750/316.2 = 68991583 mm^3

S_2 = 21813503750/1215.8 = 17941338 mm^3

A (steel section only) = 25500 mm^2

A_t = 2400 mm^2

A_w = 15600 mm^2

A_b = 7500 mm^2

A_c (Transformed section) = 80000 mm^2

Eccentricity of tendon e = 1215.8-100 = 1115.8 mm

(a) Stress at top $f_{cp}^{t,4}$ = $-M_{LL+Impact}/S_1$

= -1800*1e+06/68991583.2

= -26.09 N/mm^2 (in steel units) or

= -3.48 N/mm^2 (in concrete units)

(b) Stress at bottom $f_{cp}^{b,4}$ = $M_{LL+Impact}/S_2$

= 1800*1e+06/17941337.9

= 100.33 N/mm^2 (in steel units)

Numerical Examples

STAGE 5: Increment of prestressing force in the tendon due to live load + impact, n =7.5

$M_{LL+Impact+SIDL}$ = 550+1800 = 2350.00 kN-m

Increment ΔX = $2*(M_{LL+Impact+SIDL}).e*(2-L_o/L)/(3*(e^2+I_{CP}/A_{CP})+EI_{CP}/(E_tA_t))$

L_o = L for UDL

= 2*2350*1e+06*1115.8*1e-03/(3*(1115.8^2 +21813503750/105500+(200000*21813503750.2)/(160000*576)))

= 35.8 kN

(a) Stress at top $f_{cp}^{t,5}$ = $-\Delta X/A_{CP}+\Delta X.e)/S_1$

= 35.8*1e+03/105500+35.8*1e+03*1115.8/68991583.2

= 0.9 N/mm² (in steel units) or

= 0.1 N/mm² (in concrete units)

(b) Stress at bottom $f_{cp}^{b,5}$ = $-\Delta X/A_{CP}-\Delta X.e)/S_2$

= 35.8*1e+03/105500+35.8*1e+03*1115.8/17941337.9

= 2.6 N/mm² (in steel units)

STAGE 6: Final Stresses

(a) Stress at top Σf_{CP}^{t} = -2.34-0.88+0.12-3.48 = -6.6 N/mm² (in concrete units)

(b) Stress at bottom Σf_{CP}^{b} = +56.28+31.98+100.33 = 188.6 N/mm² (in steel units)

Selection of tendon :

Tendon area requirement = $(X+\Delta X)/f_t$ = (547.2+35.8)*1e+03/950 = 613.72 mm²

Use BBRV - 2 Nos of cables one on each side of the web and each cable comprising of 10 wires of 7 mm dia

Final A_T = 3.1416/4*7^2*2*10 = 769.69 mm²

or use 2-12 ϕ 5 tendons one each on either side of web.

4. Check for shear :

We calculate the maximum web shear at the supports and at different positions of the neutral axis corresponding to long & short term loads.

Total shear at support V_{max} = 760.00 kN due to all loads

(a) Web shear in steel girder composite with slab.

n =15

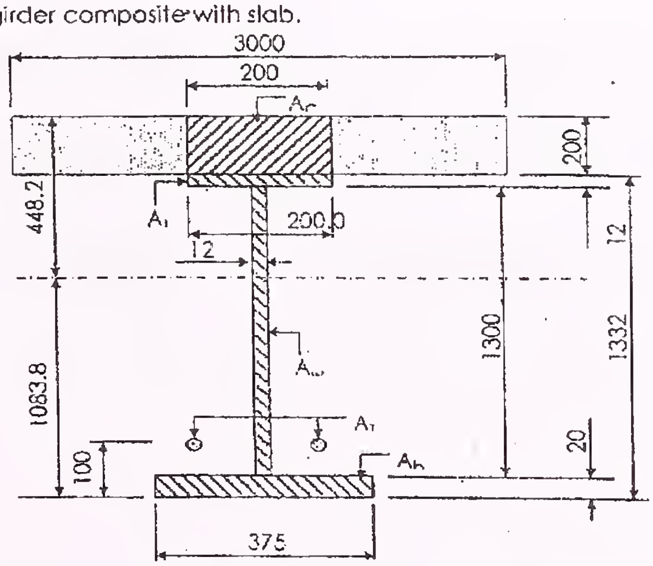


Fig A3.06

Q_{NA}^{top} = 200*200*348.2+12*200*242.2+236.2*12*-106.0 = 14208520 mm³

Q_{NA}^{bot} = 375*20*1073.8+1063.8*12*531.9 = 14843677 mm³

Therefore τ_{NA} = 760*1e+03*14843677.3/(18640170910*12) = 50.4 N/mm²

(b) Web shear in steel girder composite with slab.

n = 7.5

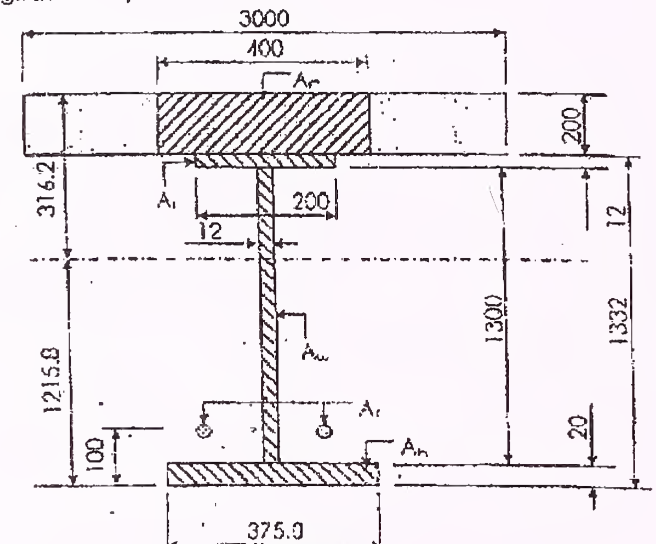


Fig A3.07

Q_{NA}^{top} = 400*200*216.2+12*200*110.2+104.2*12*-106.0 = 17426015 mm³

Q_{NA}^{bot} = 375*20*1205.8+1195.8*12*597.9 = 17623644 mm³

Therefore τ_{NA} = 760*1e+03*17623643.6/(21813503750*12) = 51.2 N/mm²

5. Deflection :

(a) Due to prestressing force δ_{or} = (547.2+35.8)*1e+03*983.8 *24000^2/(8*200000 *18640170909.5) = 11.1 mm upward

(b) Due to DL & SIDL $\delta_{DL+SIDL}$ = 2200*1e+06*24000^2/(10*200000*18640170910) = 34.0 mm downward

Camber to be provided : 20.0 mm upward

(c) Due to LL+Impact $\delta_{LL+Impact}$ = 1800*1e+06*24000^2/(10*200000*21813503750) = 23.8 mm downward

Net deflection δ = 23.8+34.0-11.1-20 = 26.7 mm downward

And, $\delta_{LL+Impact}$/span = 23.8/24000 = 0.0010

< limiting deflection, OK

Numerical Examples

Prestressed Steel Road Bridges

Numerical Example 3: Individual Prestressed Truss Member in tension.

Find the cross sectional area of prestressed truss member with the following data:

Tensile force F_{total} : 1700 kN
Allowable stress in the member f_m = 150 N/mm^2 (Mild Steel)
Allowable stress in the high strength steel tendon bar f_t = 950 N/mm^2
The co-efficients are:

k =	5	ψ =	0.9	β =	1
		n_1 =	1.1	n_2 =	0.9

where
- k = $A_t/\Sigma A = A_t/(A_t+A_m)$
- A_t = c/s area of prestressing tendon
- A_m = c/s area of steel member
- ψ = longitudinal bending co-efficient of steel member (used in the range of 0.9-0.95)
- n_1 = over loading factor under prestressing force
- n_2 = factor for reduction of the actual prestressing force (loss due to relaxation of steel & yielding of anchorage)

Note: When safe and direct control of prestressing is assured then during design of the member having a tendon of steel cable, the values of these coefficients are n_1 = 1.05 and n_2 = 0.95.

Step 1. The required cross sectional area of tendon (A_t) is given by:

$$A_t = \frac{n_1\psi\beta\alpha^2 f_m}{\alpha(\beta f_t+n_1\psi f_m)-F_{total}} \rightarrow \frac{n_1\psi\beta F_{total}}{f_m(1+n_2\psi)(\beta k+(n_1-n_2)\psi-1)}$$

or

$$A_t = \frac{1.1*0.9*1*1700*1e+03}{150*(1+0.9*0.9)*(1*5+(1.1-0.9)*0.9-1)} = 1482.99 \text{ mm}^2$$

Step 2. The required cross sectional area of the member is given by :

$$A_m = \alpha - A_t/\beta = F_{total}/(f_m(1+n_2\psi)) - A_t/\beta =$$

$$= \frac{1700*1e-03}{150*(1+0.9*0.9)} - \frac{1482.99}{1} = 4778.52 \text{ mm}^2$$

use the cross section of the member as 2 channels say 2MC 200
having total cross sectional area = 5642 mm^2 ; and the cross section of the tendon from two high-strength steel bars, each having diameter D = 32 mm
Therefore A_t = 2*3.1416*32^2/4 = 1608.50 mm^2

Step 3. The force due to initial prestressing of the tendon may be found from the expression:

$Z = \psi f_m A_m$ = 0.9*150*5642.00*1e-03 = 761.67 kN

Step 4. To check the correct choice of the cross section of member, substitute the found values of Z, F_{total}

$\Delta F_{total} = F_{total} \cdot A_t/(A_t+\beta A_m)$ = 1700*1608.50/(1608.50+1*5642) = 377.14 kN

$Z*n_1+\Delta F_{total}$ = 761.7*1.1+377.14 = 1214.98 kN

$f_t.A_t$ = 950*1608.50*1e-3 = 1528.07 kN

So, $Z*n_1+\Delta F_{total}$ < $f_t.A_t$ OK

Step 5. $-Z^{*}\eta_2+(F_{total} - \Delta F_{total}) = -761.7^{*}0.9+(1700-377.14) =$ 637.36 kN

$f_mA_m =$ $150^{*}5642^{*}1e\text{-}03 =$ 846.3 kN

So,

$-Z^{*}\eta_2+(F_{total} - \Delta F_{total}) < f_mA_m$ OK

Step 6. The reduction of the total cross sectional area of the member due to the prestressing is:

$$\frac{F_{total}/f_m - (A_m+A_t)}{F_{total}/f_m} {}^{*}100 = \frac{1700^{*}1e{+}03/150-(5642+1608.50)}{1700^{*}1e{+}03/150} {}^{*}100$$

$$= 36.03\ \%$$

Step 7. To secure the value of the co-efficient $\psi = 0.9$, it is necessary to install diaphragms between both channels considering lateral flexibility of the channel. For $f_m = 150N/mm^2$, the value of flexibility co-efficient $\lambda = 40$. However,considering that the diaphragm has a somewhat larger diameter of the hole than the diameter of the prestressing bar, the actual flexibity proposed is as $0.8\lambda = 32$.
Therefore, the transverse diaphragms should be spaced at $a = 32r_y$ where r_y is the minimum radius of the gyration of a single channel.

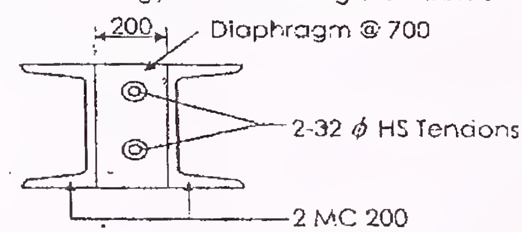


Fig A3.08

Numerical Examples

Prestressed Steel Road Bridges

Numerical Example 4: Prestressed Truss

Design a truss with its bottom chord prestressed and under an uniformly distributed load of q = 37.5 kN/m. The tendons are at the axis of the bottom chord connected at locations between sections 2 & 5 (Figure 1).

E_m = 200000 N/mm² E_t = 160000 N/mm²

For the tendon wires use 7 mm wires with f_t = 950 N/mm² and

for steel members f_m = 150 N/mm² (mild steel)

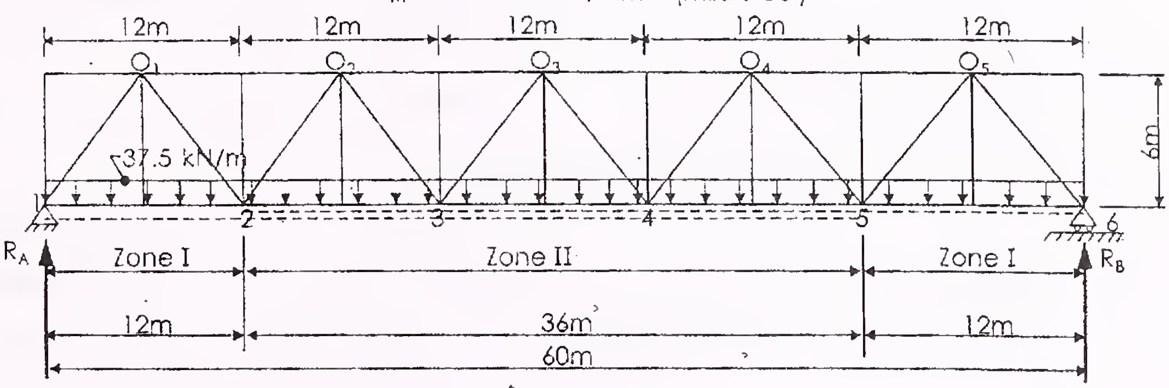


Fig.A3.09

1. Axial tensile forces at the bottom of the non-prestressed truss (for bottom chord) area follows:

Reactions : R_A = 37.5*60/2* = 1125 kN = R_B

For Panel 1-2: M_{O1} = 1125*6-0. 375*6^2/2-F1*6 = 0
or F_1 = 1/6*(1125*6-37.5*6^2/2) = 1012.5 kN

similarly :

For Panel 2-3: F_2 = 1/6*(1125*18-37.5*18^2/2) = 2362.5 kN

For Panel 3-4: F_3 = 1/6*(1125*30-37.5*30^2/2) = 2812.5 kN

2. The cross section of the bottom chord is selected as 2 MC 400, having cross sectional area 6500 mm² each giving a total area of 13000 mm²

3. Tendons

(a). Zone I : A_t^{I} = 25 Nos. of 7 mm wires 962.11 mm²

(b). Zone II : A_t^{II} = 50 Nos. of 7 mm wires 1924.23 mm²

4. The increase of the prestressing in the tendon force will be found by equalizing the elongation of the bottom chord to the elongation of the tendon, namely :

(a). Elongation of the bottom chord in Zone I:

$$\Delta l_1 = \frac{(F_1 - n_1\Delta X)*a}{E_m A_1}$$

(b). Elongation of the bottom chord in Zone II:

$$\Delta l_2 = \frac{[(2F_2+F_3)-n_2 2\Delta X]*a}{E_m A_2}$$

(c). Elongation of the tendon:

$$\Delta l_t = \frac{n_1 a\Delta X}{E_t A_t} + \frac{n_2 a 2\Delta X}{E_t 2A_t} = \frac{(n_1+n_2)a\Delta X}{E_t A_t}$$

where n1, n2 = number of panels in Zone I and Zone II respectively.
a = panel length

(d). For compatibility - $\Delta l_t = \Delta l_1 + \Delta l_2$

i.e. $$\frac{(n_1+n_2)a\Delta X}{E_t A_t} = \frac{(F_1-n_1\Delta X)*a}{E_m A_1} + \frac{[(2F_2+F_3)-n_2 2\Delta X]*a}{E_m A_2}$$

Numerical Examples

where, $\Delta X = \dfrac{F_1/A_1+(2F_2+F_3)/A_2}{n_1/A_1+2n_2/A_2+(n_1+n_2)/A_1{}^*E_m/E_t}$

$= \dfrac{1012.5^*1e+03/13000+(2^*2362.5+2812.5)^*1e+03/13000}{1/13000+2^*3/13000+(1+3)/962.11^*200000/160000}$

$=$ 114673.3 N = 114.67 kN

5. The permissible force in one tendon X_0 = 962.11*950/1000 = 914.01 kN

6. $X_0 = X + \Delta X$ where X = Prestressing force $= X_0 - \Delta X$ 799.33 kN

7. Stress in the tendon is :

   $\sigma_t = X_0/A_t = (X+\Delta X)/A_t =$ 950 N/mm^2 $\leq f_t$, OK

8. Stress in member 3-4 with all loads applied

   $\sigma = \dfrac{F_{34} - 2^*(X+\Delta X)}{A_2} = \dfrac{2812.5^*1e+03-2^*(914.01^*1e+03)}{13000} =$ 75.73 N/mm^2

   $< f_m$, OK Tension

9. Stress in member 3- during prestressing without applied loads :

   $\sigma = \dfrac{2X_0}{A_2} = \dfrac{2^*914.01^*1e+03}{13000} =$ 140.62 N/mm^2 $< f_m$, OK Compression

Deflections

10.

Member	Forces	1 - 2	2 - 3	3 - 4	4 - 5	5 - 6
Force $S_{i,x}$ in member	Due to force in tendons	1012.5	1012.5	2362.5	2362.5	2812.5
Force $S_{i,i}$ in member	Due to unit vert. Force at center	0.5	0.5	1.5	1.5	2.5

$\Sigma (S_{i,i}S_{i,x}l_i / E A_i) =$ 69.8 mm which is L/857 OK

(The Official amendments to this document would be published by the IRC's in its periodical, 'Indian Highways', which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified there in)